THE EFFECT OF LEFT TURN SLIP LANES ON SAFETY AND OPERATIONAL PERFORMANCE OF SIGNALISED INTERSECTIONS

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Abstract

There are a number of arguments about the safety of using left turn treatments, particularly left turn slip lanes, at signalised intersections. Some suggest that left turn slip lanes might be a safer facility for all road users especially pedestrians. On the other hand, other arguments are not in favour of using them.

There is limited research available in that area around the world, especially in New Zealand. Thus, the main aim of this thesis was to evaluate the effect of left turn slip lanes on the safety performance and the operational performance at signalised intersections in New Zealand.

The safety performance of 625 signalised intersections in Auckland, including 1818 approaches, were evaluated using three key analyses: overall crash analysis, detailed crash analysis, and additional detailed pedestrian crash analysis.

The results were examined and were statistically verified using the Chi-square test. It was found that the frequency of left turn crashes was minimal, especially pedestrian crashes. Furthermore, the greatest proportion of the left turn crashes was non-injury.

In terms of safety performance, both left turn slip lanes and left turn conventional lanes have similar safety performance. The largest proportion of pedestrian crashes and injuries occurred predominantly at the shared conventional lanes and at the zebra crossing slip lanes, thus making them the least options to be chosen when designing signalising intersections.

The operational performance was carried out by modelling 24 scenarios to assess the intersection performance for left turn slip lanes versus conventional lanes. The key parameters used were delay, queue lengths, and level of service. Further analysis was conducted testing a range of left turn flows, intersection flows and applying different pedestrian protection times. This was to determine the wider implications on the intersection operational performance as a result of those factors.

The results showed that the left turn slip lanes contribute to the resilience of the intersection performance, even with increasing the traffic flow of the left turn movements and/or of the whole intersection. The use of left turn slip lanes can significantly reduce delays experienced by left turning traffic movements, to their relevant approaches and the overall intersection delay. On the contrary, the left turn conventional lanes, especially shared lanes, contributed immensely to the increase of the delay to these movements, to their approaches and to the delay of the intersection as a whole.

The thesis was concluded by a set of recommendations for the safety performance and the operational performance of left turn slip lanes in comparison with left turn conventional lanes, at signalised intersections.

Glossary of Terms, Abbreviations and Acronyms

Definition of terms as they used in this thesis:

Terms	Definition
AADT	Average annual daily traffic volume.
ADT	Average daily traffic volume.
Calibration	It is the process of adjusting operational parameters and settings within the model so the outputs are a suitable representation of observed conditions.
Capacity	The theoretical maximum volume of traffic that a particular intersection movement can accommodate.
CAS	Crash Analysis System is New Zealand's primary computer system for capturing information on where, when and how road crashes occur. It also provides tools to collect, map, query and report on road crashes and other related data.
CIS	Controller Information Sheets contain the information used by the software specialist to generate specific traffic signals controller software program.
CL or CT	Cycle Length or Cycle Time is the time required for one complete sequence of signal displays (sum of phase green and intergreen times) for a given movement. The phase time is the sum of the durations of red, yellow and green signal displays; the cycle time is the sum of the phase times.
Clearance Time	Time given to allow a terminating movement of vehicles or pedestrians to vacate the controlled area, before the beginning of the next movement of traffic.
DOS	Degree of Saturation is the ratio of the traffic demand (ie traffic volume or flow rate) to the theoretical capacity of the intersection.
Left Turn Slip Lane or Channelised Left Turn Lane	The term "left turn slip lane" or "channelised left turn lane" are used interchangeably in countries that drive on the Left Hand Side. On the contrary, the term "right turn slip lane" or "channelised right turn lane" are used in countries that drive on the Right Hand Side. In the literature review of the research, the terms "slip lane" and "channelised lane" are used alternately, otherwise in the rest of the research, the term left turn slip lane is used.

Peak Period	The time of day when traffic demand is at a maximum, e.g. morning (AM Peak) and evening (PM Peak) for commuter work trips. Other times are called Off Peak periods.
Pedestrian Clearance Time	It is The Flashing Don't Walk period that immediately follows the termination of pedestrian Walk display to enable pedestrians, who have just stepped off the kerb at the commencement of this period, to complete their crossing to the nearest kerb.
Pedestrian protection times	It is holding the left turning traffic on red signal (red arrow display) for a period of time to ensure drivers are forewarned of the potential pedestrian conflict.
Signal Phase	A traffic signal state during which one or more vehicle movements receive right of way (i.e. green signal display or arrow signal display).
Phase Sequence	The order of phases in a signal cycle.
Phase Split	Phase Split is the duration of each phase (green time and intergreen time) within a signal cycle. It is normally expressed as a percentage of cycle length.
Phasing	A pre-set order of traffic signal phases and the time allocated to each one.
QGIS	Quantum Geographic Information System. It is an open source Geographic Information System that supports most geospatial vector and raster file types and database formats.
RAMM	Road Asset and Maintenance Management is an Internet accessible software database system that stores the traffic signal assets.
SCATS	Sydney Co-ordinated Adaptive Traffic System. It is a fully adaptive area wide control system developed by Road and Maritime Services Roads (RMS) (formerly called Roads and Traffic Authority of New South Wales, RTA) which used to co-ordinate and monitor traffic signal intersections in New Zealand.
SIDRA	Signalised and unsignalised intersection design and research aid. The analytical traffic analysis software developed by the Australia Road Research Board.
Slip Lane	An area of carriageway for vehicles turning left that is separated, at some point, from other parts of the road by a triangular traffic island.

TCRs	Traffic crash reports are completed by police officers at the scene of all road crashes, including non-injury crashes. They contain extensive data that detail exactly where, when, how and why the crash happened.
TGSI	Tactile Ground Surface Indicators provide pedestrians with visual and sensory information. The two types of TGSI are warning indicators and directional indicators. Warning indicators alert pedestrians to hazards in the continuous accessible path of travel indicating that they should stop to determine the nature of the hazard before proceeding further.
Intergreen Time	Intergreen Time is the sum of the Yellow and All-Red Times which is the clearance interval at the end of the signal phase.
ALL-Red Time	All-Red Time is the time between the end of the yellow signal on one phase and the commencement of the green signal on the next phase.

Units:

km/h	kilometres per hour
s/veh	seconds per vehicle
vpd	vehicles per day
vph	vehicles per hour
S	seconds

Abbreviations used in Figures, Tables and Graphs:

LT	left turn
SE	South East Approach
NW	North West Approach
SW	South West Approach
Rd	Road
St	Street
Ave	Avenue
VS	Versus

1 Introduction

Generally, turning movements have an effect on the safety and the operational performance at signalised intersections. Therefore, various treatment methods for turning movements have been considered in the design and operational analysis of signalised intersections. Historically, the right turn movement at signalised intersections has received much more attention than the left turn movement (Perez, 1995).

Left turn treatments have been considered in the design of signalised intersections by most transportation engineers, especially left turn slip lanes. However, the impact of left turn slip lanes on safety for motorists and pedestrians has not been clearly studied. There is no crash historical data or established methodology available to evaluate the safety and operational performance of left turn slip lanes. To date, research offers at best minimal evaluation of left turn treatments at signalised intersections.

The key objective of this research is to investigate the effect of left turn slip lanes on safety performance and the operational performance at signalised intersections. This research was undertaken in New Zealand during the period 2015–2016.

The term “left turn slip lane” or “channelised left turn lane” are used interchangeably in countries that drive on the Left Hand Side. On the other hand, the term “right turn slip lane” or “channelised right turn lane” are used in countries that drive on the Right Hand Side.

In the literature review of the research, the terms “slip lane” and “channelised lane” are used alternately. Otherwise in the rest of the research, the term “left turn slip lane” is used.

1.1 Left turn treatment types

Left turn treatments for signalised intersections are generally categorised into two main types: conventional lanes and slip lanes. These two main categories include seven subcategories as depicted in Figure 1-1. They are listed as follows:

Conventional left turn lane

- Shared conventional lane (shared through and left turn lane, or shared left and right lane);
- Exclusive conventional lane.

Slip left turn lane

- Signal control slip lane, with signalised pedestrian crossing marking;
- Give-way control slip lane, with no pedestrian crossing marking;

- Zebra crossing control slip lane, with zebra pedestrian crossing marking;
- Zebra crossing control slip lane, with zebra pedestrian crossing marking on *raised table*; and
- Free flow slip lane, with no control.

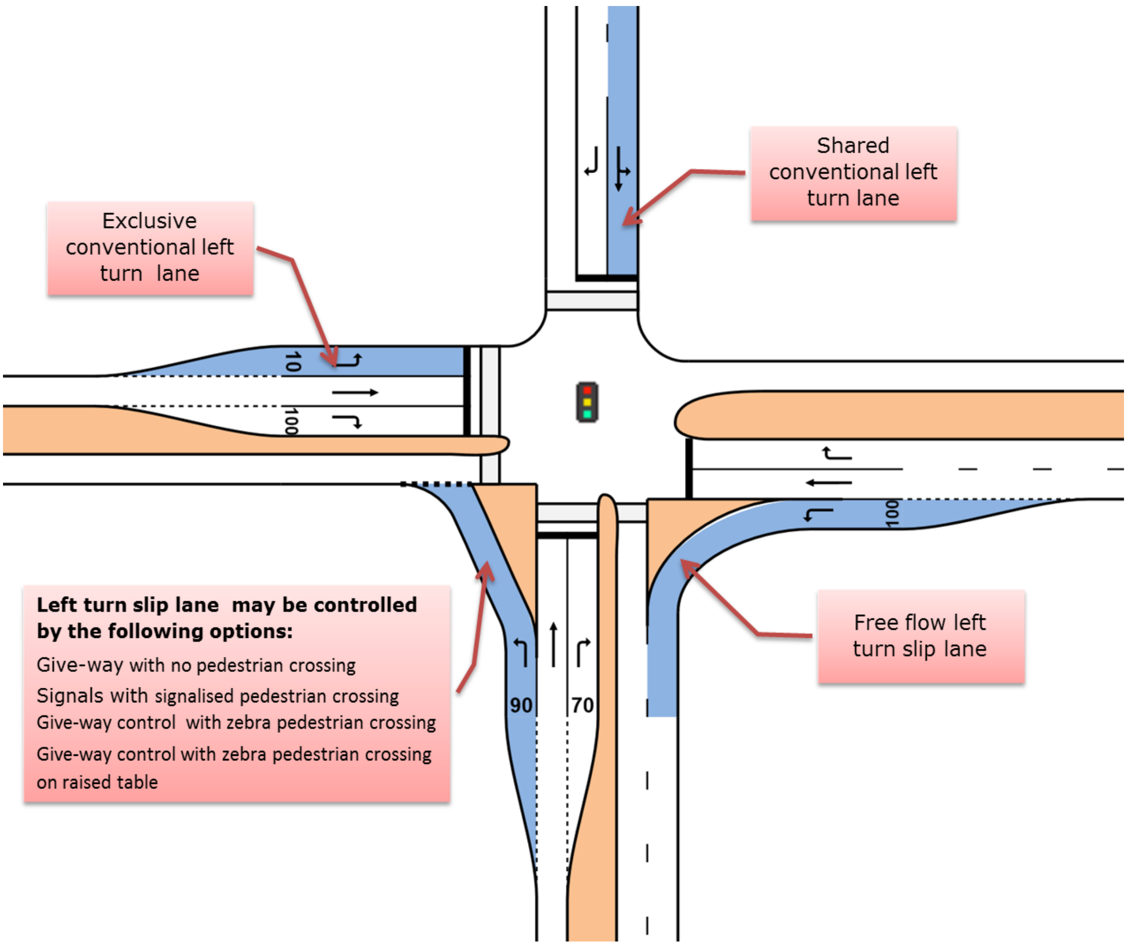


Figure 1-1 Different left turn treatments used at signalised intersections

1.2 Research motivation

Currently among transportation engineers in the industry, there are a number of arguments regarding the safety of left turn slip lanes at signalised intersections. This issue is especially related to pedestrians. There are a number of advocates who state that left turn slip lane treatments may be problematic and can cause a safety issue for pedestrians. Conversely, there are some who are in a favour of such treatments. They think it may provide a safer facility for most pedestrians rather than the conventional left turn treatments. From their perspective, it offers better and overall balanced safety and operational outcomes for all road users. This raises the main research objectives.

1.3 Research objectives

The purpose of this research was to assess the safety performance and the operational performance of left turn slip lanes versus conventional lanes. This is to provide evidence based on data to support or oppose the above argument. The key objectives of this research are summarised as follows:

Literature Review

1. To review all relevant literature relating to the safety and the operational performance of signalised intersections involving various left turn treatments;

Evaluation of the Safety Performance

The key objective was to evaluate the safety performance of left turn slip lanes versus left turn conventional lanes in the signals network that was conducted as follows:

2. The **overall crash analysis*:*** To compare the frequency, severity, crash movement codes, contributing factors of crashes involving left turn movements that occurred at *signalised intersections approaches* for various left turn treatments, in the entire network for a period of **one year;**

3. The **detailed crash analysis**: To compare in depth the frequency and severity of crashes involving left turn movements that occurred at *selected signalised intersections approaches* for various left turn treatments for a period of **five years**;

4. Additional **pedestrian crash analysis**: To compare in depth the frequency, severity, and crash movement codes of left turn crashes involving pedestrians that occurred at *signalised intersections approaches* for various left turn treatments, in the entire network for a period of **five years**;

Evaluation of the Operational Performance

5. To assess the operational performance of left turn slip lanes treatments versus left turn conventional lanes using intersection modelling, in terms of delay, queue length and level of service; and

6. Further analysis to determine the implications for the intersection operational performance as a result of increasing the different range of left turn flows, intersection flows, and pedestrian protection times.

1.4 Structure of this report

This research contains the following chapters detailing the work undertaken. Following a review of the literature (Chapter 2), a desired methodology was developed and data collection was completed (Chapter 3). An overall crash analysis undertaken (Chapter 4), and a detailed crash analysis was conducted (Chapter 5). An additional detailed pedestrian crash analysis was carried out (Chapter 6). A statistical analysis was completed (Chapter 7), and intersection performance was accomplished (Chapter 8). Finally, conclusions and recommendations for further research were made (Chapter 9).

2 Literature Review

2.1 Introduction

The purpose of this chapter is to provide an overview of different left turn treatments and particularly left turn slip lanes at signalised intersections. It explores design elements and guidance provided in national and international reference material. It is then followed by a detailed review of national and international studies related to the safety and operational aspects of differing left turn treatments, including left turn slip lanes.

It is worth mentioning that most of the studies reviewed in this literature originated from various countries where driving rules, design standards and terminologies may differ from those used in New Zealand. In particular, the following main differences should be considered:

- Right-side driving versus left-side driving;
- Yielding versus give-way; and
- Urban traffic control systems: SCATS® versus others systems.

To date there is limited research in New Zealand and Australia which has studied left turn slip lanes or different left turn lane treatments based on data analysis assessing and comparing the safety, and operational performance of individual treatments. On the other hand, a number of research studies were found in the United States relevant to left turn slip lanes.

It is commonly believed that the installation of left turn slip lanes improves the safety of motor vehicles and increases operational efficiency, but only limited quantitative data is available to demonstrate this theory. In addition, there is limited research found relating to pedestrian safety at left turn slip lanes.

There are many guidelines documenting the design of different left turn treatments; however, there is currently very little content on the safety and operational effectiveness of these treatments at signalised intersections. These guides provide information on available left turn treatments, including left turn slip lanes but little or no research on the safety and the operational benefits/disbenefits of each type of facility has been provided to date.

2.2 Design of left turn slip lanes

This section provides an overview of the existing standards, guidelines, manuals and key relevant reference material that are available in New Zealand, Australia, the United Kingdom and the United States.

2.2.1 Left turn slip lanes in New Zealand and Australian Contexts

2.2.1.1 Purpose of the left turn slip lanes

The NZ Transport Agency (2010) indicated the purpose and requirements of left turn slip lanes at signalised intersections; it stated that the left turn slip lanes are provided at an intersection to improve safety, minimise delays to through vehicles or to ease the left turn movement where the angle of the intersection would result in an otherwise difficult movement. However, currently no studies based on crash data or operational performance, are available to support these assumptions.

Austroads (2007) stated that slip lanes may be provided for heavy left turn movements at signalised intersections in urban areas to improve the level of service. It also provides the key detailed design elements of the left turn slip lane, its associated left turn island and various slip lane layouts.

Wilke (2006) reviewed and summarised the findings of a number of traffic signal audits carried out in New Zealand. One of the issues covered in this report is the left turn slip lane. The auditor examined specific design considerations that related to left turn slip lanes and recommended best practice treatments for signalised intersections. In addition, the author highlighted the key benefits of slip lanes at signalised intersections as follows:

- They help to make intersections more compact, and enable traffic signal poles and signal lanterns to be placed closer to drivers' line of sight;
- They generally simplify the decision-making processes for motorists, resulting in a safer intersection layout;
- While some pedestrians voice reservations about slip lanes, they do remove the conflict that occurs when left turners and parallel pedestrians proceed together; and
- They help cyclists to manage conflict between left-turning motorists and straight-through cyclists. Slip lanes are a good tool for achieving this, especially with the use of a coloured surface for the cycle lane over which left turners must cross.

However, the benefits listed above were not supported by any studies or statistics.

The NZ Transport Agency (2009) outlined the benefits of left turn slip lanes in managing heavy vehicle conflict with pedestrians at signalised intersections. It explained that the presence of a left turn slip lane improves intersection safety and efficiency for all road users. Again, there were no studies, based on crash history or operational performance available to support these assumptions.

2.2.1.2 Type and layout of left turn slip lanes

The NZ Transport Agency (1993) identified preferred practice for the signing and layout of slip lanes at signalised intersections in New Zealand. The key summary points provided in this guideline were:

- There are two distinct types of left turn slip lanes:
  - Free flow slip lane: The free flow slip lane is characterised by an exclusive merge/acceleration lane; and
  - High entry angle slip lane: The high entry angle slip lane is characterised by the lack of an exclusive merge/acceleration. It can be controlled by two ways:
    - High entry angle slip lane with give-way control; and
    - High entry angle slip lane with traffic signal control.

  It is recognised that many existing left turn slip lanes in New Zealand do not conform to the design of the free flow slip lane type or the high entry angle type;
- Pedestrian zebra crossings across slip lanes should only be marked if the pedestrian crossing warrant is met;
- Pedestrian crosswalk lines should be installed only when a pedestrian phase is provided on a traffic signal controlled left turn slip lane; and
- Dropped kerb crossings should always be provided where pedestrians are intending to cross a left turn slip lane.

The NZ Transport Agency (2010) listed three types of left turn slip lane designs, including detailed drawings of marking and signing plans as provided in Figure 2-1, Figure 2-2 and Figure 2-3 respectively:

- Left turn lane with give-way control;
- Left turn lane with pedestrian crossing (zebra crossing); and
- Left turn lane with signal control.

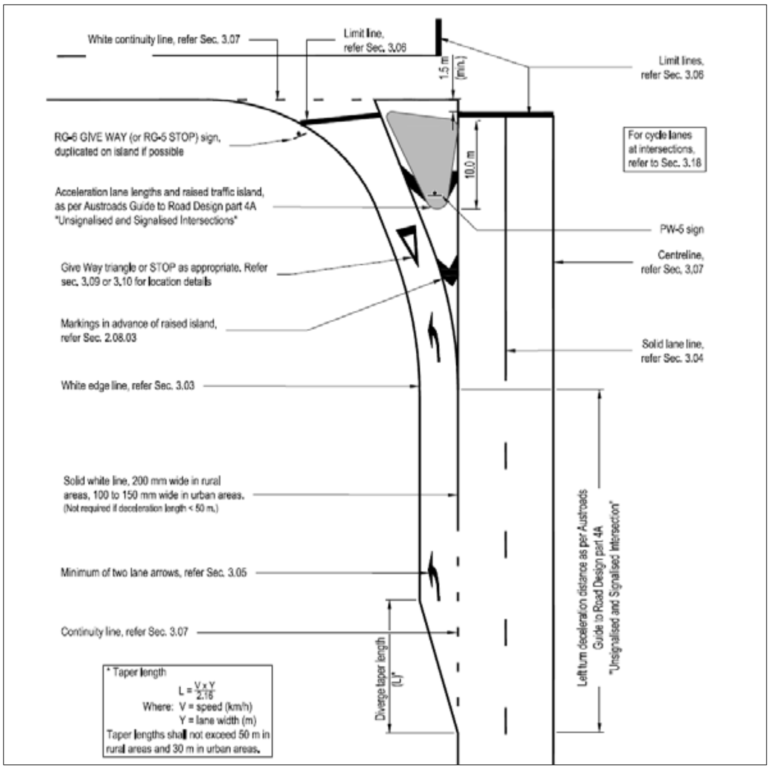


Figure 2-1 Layout for left turn slip lane with give-way control

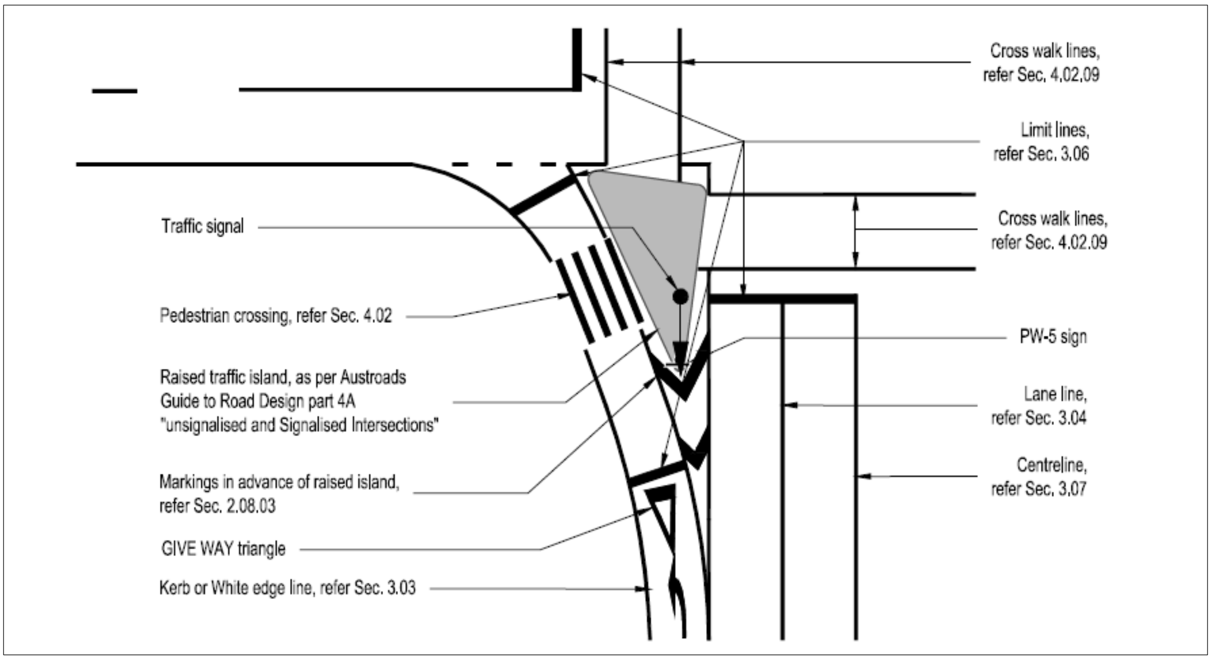


Figure 2-2 Layout for left turn slip lane with zebra pedestrian crossing

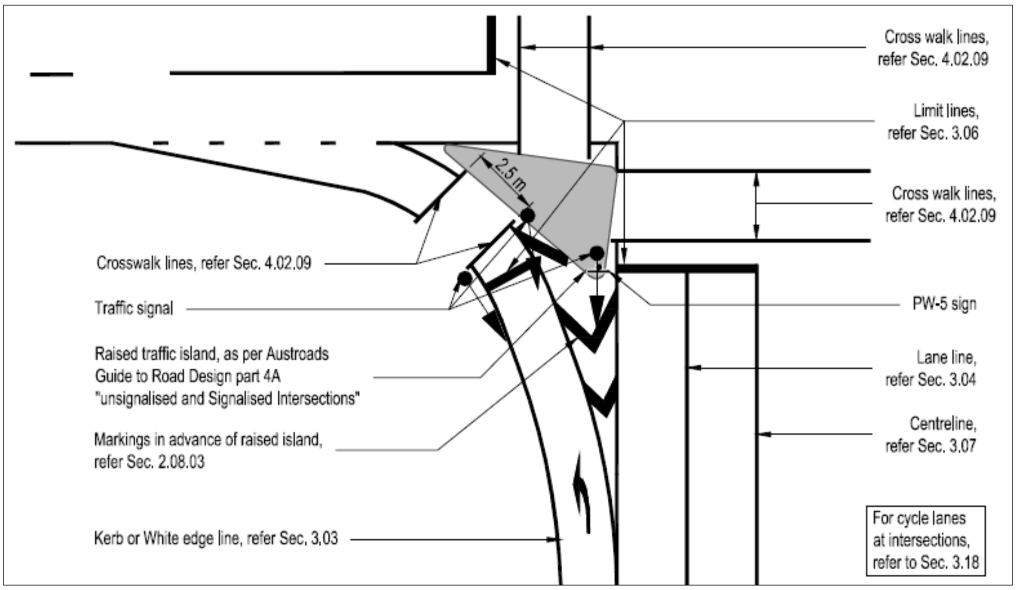


Figure 2-3 Layout for left turn slip lane with signalised pedestrian crossing

Austroads (2009a) explains varying types of left turn treatments and provides guidance on situations and factors for their use. It provides detailed design elements for two types of left turn slip lanes: high entry angle and free flow. Essentially there are three types of left turn treatments available at signalised intersections, namely:

- ❖ Basic left turn treatment, where turning vehicles may share the lane with through traffic movements, used on major and minor roads;
- ❖ Auxiliary lane left turn treatment, where a separate lane is provided to enable the turn to be performed in an additional lane;
- ❖ Channelised left turn treatment, which provides a traffic island to enhance the safety of left-turning vehicles.

The type of left-turn treatment used may depend on the following factors:

- ❖ Volume and type of traffic making the left turn;
- ❖ Volume, speed and type of traffic with which the turn merges;
- ❖ Estimated speed at entry, and desirable speeds through and exiting from the turn;
- ❖ Site restrictions such as turn angles, property boundaries, service utilities and other structures; and
- ❖ Provision for turning cyclists and pedestrian movements.

These factors combine to determine the type of treatment to be applied in a specific situation. Figure 2-4 shows an example of a left turn slip lane designed as a high entry angle, with and without an approach cycle lane.

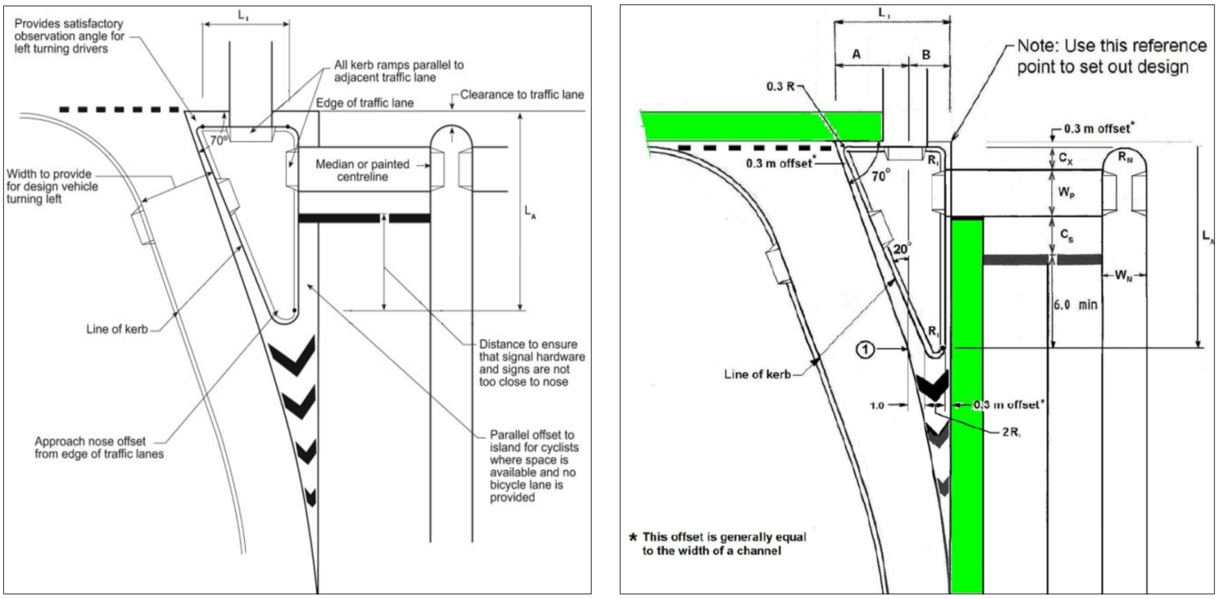


Figure 2-4 Example of a high entry angle left turn slip lane treatment with and without cycle lane

2.2.1.3 Angle of left turn slip lane and pedestrian control

The NZ Transport Agency (2009) guide recommended that in designing slip lanes, it is important to have a high entry angle to reduce traffic speeds and thereby reduce the risk for pedestrians, as shown in the Figure 2-5.

Figure 2-5 Example of a left turn slip lane treatment (zebra on raised table)

The document points out that in slip lanes there is only a single slip lane to cross. Therefore, the crossing opportunities will be frequent unless traffic flows are very high, so that the kerb crossings alone will often be sufficient. However, if pedestrian priority is highly desired, then the use of a zebra crossing, or zebra crossing on a platform should be considered.

Where continuous streams of pedestrians are unduly interrupting left turning traffic, controlling the left turn slip lane with signals may be considered but at the expense of pedestrian delay and compliance. Finally, it is noted that vision impaired pedestrians prefer signalised slip lanes rather than other control methods.

Austroads (2009b) generally provides relevant guidance for pedestrian control at left turn slip lanes. In particular, it indicated that a typical pedestrian zebra crossing application is usually provided across left turn slip lanes. At these facilities traffic regulations require a motorist to give-way to pedestrians on the crossing.

This facility relies on the motorist seeing the pedestrian on the carriageway, and then slowing or stopping if necessary to allow the pedestrian to proceed across the roadway. The driver's obligation is to give-way, but having done so, may proceed without waiting for the pedestrian to clear the roadway. Generally zebra crossings are not favoured on arterial roads where traffic speeds and volumes are relatively high. However, they are often used to provide a formal crossing of left-turn slip lanes at signalised intersections of arterial roads.

Austroads (2007) recommended in urban signalised intersections, to design high entry angle left turn slip lanes to ensure that left turns occur at low speed, with drivers having a clear view of conflicting traffic, including any pedestrians on a zebra crossing located on the left slip lane. Where it is necessary to provide a free flow left turn lane, and pedestrians are expected to be present, an appropriate controlled pedestrian crossing should be provided.

It is noted that at higher turn radii, drivers may tend to focus on the driving task, and potentially conflicting traffic rather than pedestrians. Where significant pedestrian flows occur, turning speed may have to be controlled through road geometry.

2.2.1.4 Traffic island of left turn slip lane

The NZ Transport Agency (2015) specified the functions of slip lane islands at signalised intersections stating that they are provided to separate left turning traffic from through and/or right turning traffic; it is a place for pedestrians to wait while crossing the road. It included the detailed design and installation guideline of Tactile Ground Surface Indicator (TGSI) for visually impaired pedestrians. In addition, it recommends installing TGSI on most slip lane islands at the three crossing points as shown in Figure 2-5. This is to assist blind and vision-impaired people as well as increasing their safety while crossing roads.

Austroads (2007) highlighted that the primary function of traffic islands at the left turn slip lane is to channelise traffic into separate streams within complex intersections. Traffic islands are also used to accommodate pedestrians, traffic signal equipment, and roadway lighting. The slip lane traffic island should be provided as a raised, not a flush island, at signalised intersections. Left turn flush traffic islands should never be installed at traffic signal controlled intersections. In particular, raised traffic islands that are expected to store a considerable number of pedestrians should be placed and designed so that pedestrians are not at risk from the body overhang of large vehicles. In addition, left turn islands should be large enough to enable the correct placement of pedestrian cross walk lines, traffic signals and stop-lines.

2.2.2 Left turn slip lanes in international context

The American Association of State Highway and Transportation Officials (2004) listed the key reasons for providing a channelised right turn lane at intersections as follows:

- ❖ To increase vehicular capacity at intersections;
- ❖ To reduce delay to drivers by allowing them to turn at higher speeds;
- ❖ To reduce unnecessary stops;
- ❖ To clearly define the appropriate path for right-turn manoeuvres at skewed intersections or at intersections with high right-turn volumes;
- ❖ To improve safety by separating the points at which crossing conflicts and right-turn merge conflicts occur; and
- ❖ To permit the use of large curb return radii to accommodate turning vehicles, including large trucks, without unnecessarily increasing the intersection pavement area and the pedestrian crossing distance.

However, there is limited research to verify these benefits.

Potts et al. (2011) developed a design guide for a channelised right-turn lane at signalised intersections, and states the following:

- ❖ Kerbed islands are considered most favourable for pedestrians because kerbs most clearly define the boundary between the travelled way intended for vehicle use and the island intended for pedestrian refuge;
- ❖ Orientation and mobility specialists have a strong preference for raised islands with cut-through pedestrian paths because they provide better guidance and information about the location of the island for pedestrians with vision impairment than painted islands;
- ❖ When right-turn volumes are high and pedestrian and bicycle volumes are relatively low, capacity considerations may dictate the use of larger radii, which enable higher speed, higher-volume turns. Increasing the radius of a channelised right-turn roadway reduces right-turn delay by approximately 10 to 20% for each 5-mph increase in turning speed; and
- ❖ Small corner radii, which promote low-speed right turns, are appropriate where such turns regularly conflict with pedestrians, as higher speeds have been shown to result in a decrease in yielding to pedestrians by motorists.

Gemar et al. (2015) developed design guidelines and standard drawings for right turn slip lanes that cater to mobility as well as pedestrian and cyclist safety. These guidelines are based on synthesis findings and discussion with focus on meeting with Texas Department of Transportation (TxDOT) representatives. The guidelines were incorporated into the TxDOT Roadway Design Manual. These guidelines cover both new construction and retrofitting treatments. The key findings include:

- Road markings can be used to delineate a narrow path for vehicles and impose a sharp angle of entry into the cross street. These features promote lower-speed turns for smaller vehicles. The use of road markings enables the turning movements for heavier vehicles;
- The crosswalk should be located in the centre of turning roadway and perpendicular to it;
- Ladder pattern crosswalk markings are recommended as the transverse markings delineate the crossing location and help pedestrians with visual impairments with wayfinding, while the longitudinal markings enhance the visibility of the crosswalk for motorists;
- Deceleration lanes permit motorists to slow down before negotiating the turn and help pedestrians identify vehicles intending to enter the slip lane. On the contrary, acceleration lanes are discouraged in urban and suburban environments as they generally promote higher speeds and the slip lane becomes difficult to cross for pedestrians; and
- Placements of poles, signs, and drainage structures should avoid pedestrian walkways.

Chandler et al. (2013) provided a summary of the pros and cons of right turn slip lanes at signalised intersections, as seen in Table 2-1.

Table 2-1 A summary of the benefits and disbenefits of right turn slip lanes (Chandler et al., 2013)

Characteristics	Potential Benefits	Potential Liabilities
Safety	Separation of decelerating right-turn vehicles.	Potential for sideswipes and rear-end collisions on departure leg. Pedestrian crosswalk design compatibility.
Operations	Higher right-turn capacity. Shorter green time. Less delay for following through vehicles.	None identified. "Australian Right" may not accommodate large vehicles
Multimodal	Pedestrian refuge area.	Longer pedestrian crossing distance and exposure. Higher vehicle speeds.
Physical	Smaller impact than a lane along the right-of-way	Larger intersection footprint.
Socioeconomic	Support a mixed use, walkable community	Right-of-way costs. Access restrictions to property.
Enforcement, Education, and Maintenance	None identified.	Higher maintenance of islands, marking, signing

Highways England (2003) indicated four reasons where the left turn slip lane with a separation island should be considered:

- ❖ The left turn traffic movement is high;
- ❖ Left turn manoeuvres for heavy vehicles need to be accommodated;
- ❖ Delay for left turn vehicles would otherwise be significant; and
- ❖ Left turn traffic capacity requirements would extend the green time required for the straight ahead traffic movement phase.

It also provided two examples of left turn slip lanes with and without a taper, as shown in Figure 2-6. The first design is to be used when the proportion of the left turning traffic is low and heavy vehicles are expected infrequently. The second design is recommended to be used in situations where the proportion of heavy vehicles is large, to allow for tracking. In both examples the separation island should be designed to accommodate pedestrian refuge crossing facilities and associated signal equipment. In addition, the left turn slip lane could be signal-controlled or uncontrolled.

It recognised the need for a consistent approach to the layout of the left turn slip lanes and their associated pedestrian crossing. However, it has not mentioned any safety or operational aspects of any kind for left turn treatments including left turn slip lanes.

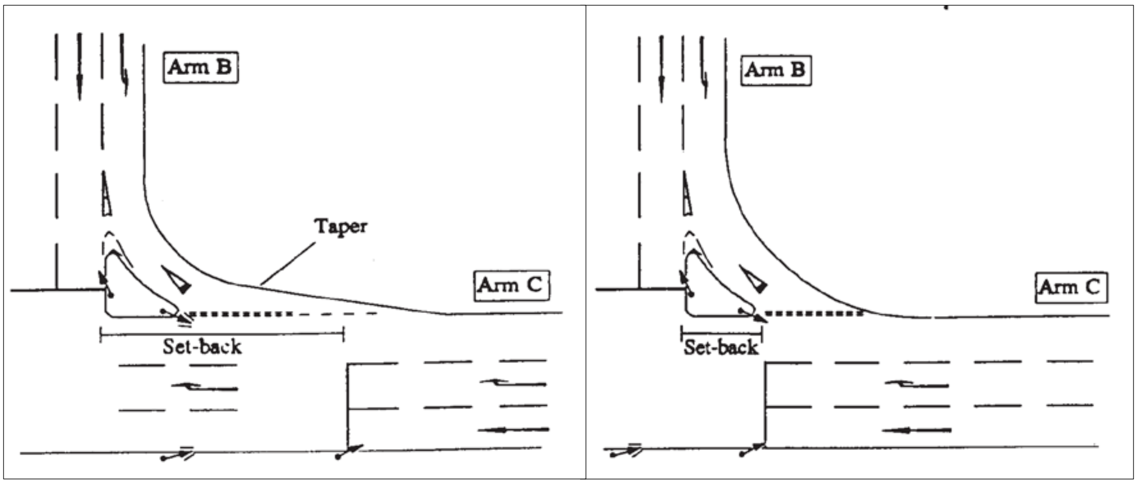


Figure 2-6 Example of a left turn slip lane with/without taper (DMRB UK, 2003)

2.3 Safety Performance of left turn lanes

2.3.1 Safety of the vehicle

Several studies compared the safety performance of different left turn treatments for motor vehicle crashes.

Dixon et al. (2000) evaluated the safety performance of various right turn treatments for 17 signalised intersections, located in County Georgia, in the metro-Atlanta area. The intersections were located on both major and minor arterials. A total of 70 right-turn approaches were identified for the study. Fifty-seven (57) of these approaches had one of the following five right-turn treatments:

- ❖ Shared right, no island, merge and no additional control;
- ❖ Exclusive right, no island, merge and no additional control;
- ❖ Exclusive right, raised island, add lane and no additional control;
- ❖ Exclusive right, raised island, merge and yield control; and
- ❖ Shared right, raised island, large turning radius, merge and yield control.

The analysis was based strictly upon crash frequencies over a 2-year period (October 1996 to September 1998) and did not include exposure data related to traffic volumes which were identified as major limitations in this study.

Unfortunately, the study was focused on vehicle versus vehicle crashes only; it did not include any pedestrian or cyclist crash data.

Table 2-2 summarises the number of right-turn crashes for each treatment.

Table 2-2 A summary of the number of right turn crashes for each treatment (Dixon et al., 2000)

Treatment	Shared right-turn lane, merge, no island, no additional control	Exclusive right-turn lane, merge, no island, no additional control	Exclusive right-turn lane, acceleration lane, raised island, no additional control	Exclusive right-turn lane, merge, raised island, yield control	Shared right-turn lane, large turning radius, merge, raised island, yield control
Number of sites evaluated	29	8	5	7	8
Number of right turn crashes for 2-year period	18	13	14	22	10
Average number of right turn crashes per site per year	0.31	0.81	1.40	1.57	0.63
Crash type	**Percent of Right-Turn Crashes Observed**				
Right angle	50	31	22	23	0
Rear-end	28	23	64	59	90
Sideswipe	17	31	7	18	0
Other	5	15	7	0	10

The following are general findings for the Dixon study:

- ❖ The use of a traffic islands appears to reduce the proportion of right-angle crashes;
- ❖ The addition of an exclusive right-turn lane corresponds to elevating sideswipe crashes; and
- ❖ The addition of an exclusive lane (i.e., an acceleration lane) on the cross street for right-turning vehicles does not reduce the number of rear-end crashes when no additional control is implemented.

Fitzpatrick et al. (2006) conducted a study sponsored by the Texas Department of Transportation (TxDOT), similar to that performed by Dixon. In this study, Fitzpatrick explored the safety experience of different right-turn lane treatments. Crash data (3-year period) for 30 right turn approaches, including nine signalised intersections were reviewed. The following right turn treatments were evaluated:

- ❖ Right-turn lane;
- ❖ Right-turn lane with raised island;
- ❖ Shared through-right lane; and
- ❖ Shared through-right lane with raised island.

Table 2-3 summarises the number of right-turn crashes for each treatment over a 3-year period. The values do not include consideration of right-turn volumes; however, they can provide an appreciation of the variability in the number of right-turn crashes among the different treatments.

Table 2-3 Annual number of right turn crashes for each type of treatment (Fitzpatrick et al., 2006)

Intersection	Total	Treatment: RTL w/island	RTL w/striping	Shared lane	Shared lane w/island
Number of approaches	30	14	6	8	2
Number of crashes	16	9	2	1	4
TxDOT project—Average number of right -turn crashes per site per year	0.18	0.21	0.11	0.04	0.67
Dixon project—average number of right-turn crashes per site per year	N/A	1.57	0.81	0.31	0.63

RTL = right-turn lane; SL = shared through-right lane.

The key findings in the Fitzpatrick study were:

- The majority of crashes (10 out of 16) were rear-end crashes. Of the 10 rear-end crashes, 5 crashes occurred in a right-turn lane with a raised island;
- The shared-lane configuration, not combined with an island, experienced the lowest average number of right-turn crashes per site per year;
- The right-turn lane separated by a raised island showed the highest number of crashes in Dixon's study and the second highest number of crashes in this study (TxDOT);
- Sites with islands have a higher number of crashes than sites without islands. This may be due to the fact that sites with islands have higher turning volumes and therefore the crash exposure is increased; and
- Frequencies of right-turn crashes are much higher at locations combining shared lanes and islands.

Theories on why the shared lanes or the right-turn without raised islands had fewer crashes include:

- Less surface area for crashes (due to absence of a turning roadway);
- Turns are from near 90-degree angle; and
- Lower speeds.

In both the Fitzpatrick and Dixon studies, the shared through/right-turn lane had the lowest number of crashes. However, due to the limited number of sites in either study and the lack of information on volumes in the right-turn lane, a more comprehensive research is needed to provide definitive advice on the safety effects of different right-turn treatments.

The Fitzpatrick study only considered vehicular traffic and did not consider the safety impacts to cyclists or pedestrians at right turn treatments. In addition, it did not provide an explanation of pedestrian facilities or phase operating characteristics of the intersections or pedestrian crossings.

It is worth noting that the authors Dixon and Fitzpatrick compared their studies to previous research and found that treatments with the highest number of crashes were right-turn lanes combining raised islands. They found that this type of intersection had the second highest number of crashes evaluated in their relevant studies.

They recommended that these findings be verified through the use of a larger, more comprehensive study that included right turning volume.

Bauer and Harwood (1996) used statistical modelling with negative binomial regression on a total of 14,432 signalised and unsignalised intersections (on rural and urban roads) in California. The study found that right-turn channelisation resulted in an increase in total multiple-vehicle crashes and injury crashes.

2.3.2 Safety of pedestrian

The following is a presentation of a few research summaries addressing the safety aspect of pedestrians at left turn treatments.

O'Brien et al. (2010) evaluated pedestrian safety for differing left turn treatments at signalised intersections located across Metropolitan Melbourne, Australia, in a study conducted for VicRoads (the State Road Transportation Agency). In particular, this study looked into left turn vehicle (including tram, bicycle and motorcycle) versus pedestrian crashes for all signalised intersections in Melbourne.

The study included large samples of signalised intersections (2,284) and a longer crash observation period (5-year period ending in 2008). The study compares the proportion of crashes occurring at each type of left turn treatment, versus the proportional use of this treatment at all signalised intersections on the Melbourne Metropolitan road network. It was considered that the volume-based crash rates were impractical due to the large sample size.

The key objectives of the study were to:

- ❖ Examine the detailed characteristics for each crash; and
- ❖ Propose a design response to the identified safety issues.

The crashes involving pedestrians have been investigated for 6,978 approaches for various left turn treatments. The left turn treatments were grouped into two major types:

- ❖ Stand-up lanes (they are conventional left turn lanes such as exclusive and shared-left turn lanes); and
- ❖ Slip lanes (signalised 2 or 3 aspects, unmarked, zebra, and free slip).

The summary of left-turn treatments, associated lane configurations and crash statistics analysis for this study is shown in Table 2-4.

Table 2-4 Pedestrian crashes and treatment frequency at signalised intersections by left turn type (O'Brien et al., 2010)

		Crashes		Injuries					Left-Turn Treatments		Crash & Injury Rates	
Lane Type	Left Turn Type/Layout	No.	% Crashes by Treatment	Fatal	Serious	Other Injury	Total	% of Injuries by Treatment	No.	% in Network	Crashes / 1000 sites	Injuries / 1000 sites
Exclusive Stand-Up Lane	L	33			11	24	35					
	L, L	3			1	3	4					
	sub-Total	**36**	**18%**	**0**	**12**	**27**	**39**	**18%**	**1,407**	**20%**	**26**	**28**
Shared Stand-Up Lane	L,L+T	3			2	2	4					
	L+R	3			2	1	3					
	L+T	100		1	39	67	107					
	L+T+R	11			4	8	12					
	sub-Total	**117**	**60%**	**1**	**47**	**78**	**126**	**59%**	**3,500**	**50%**	**33**	**36**
Slip Lane	L slip signalised (2-aspect)	2	1%	0	0	2	2	1%	115	2%	17	17
	L slip signalised (3-aspect)	3	2%	0	1	2	3	1%	147	2%	20	20
	L Slip unmarked	11	6%	2	3	10	15	7%	632	9%	17	24
	L Slip zebra	26	13%		13	14	27	13%	1,142	16%	23	24
	L Slip - free slip	0	0%				0	0%	35	1%	0	0
	sub-Total	**42**	**22%**	**2**	**17**	**28**	**47**	**22%**	**2,071**	**30%**	**20**	**23**
Total		**195**	**100%**	**3**	**76**	**133**	**212**	**100%**	**6,978**	**100%**		

"L, L" = dual left-turn lanes; "L+T" = shared left-turn & through lane; "L, L+T" = exclusive left-turn lane adjacent to shared through and left-turn lane; etc

The primary findings were as follows:

- Slip lane crashes and injuries occurred in a much smaller proportion than their frequency on the network (22% versus 30%). So, slip lanes may offer safety advantages to pedestrians and should continue to be considered as a default left-turn treatment at new intersections;
- Exclusive lanes (as with slip lanes) experienced crashes and injuries in a smaller proportion than their frequency in the network (18% versus 20%), and should also be considered as a possible alternative to slip lanes; and
- Shared stand-up lanes appeared to perform poorly, experiencing significantly more crashes and injuries than their share of treatments in the network (60% versus 50%), and should be avoided where possible.

This study is considered to be the most relevant study to this research as VicRoads has similar traffic signal operating system (SCATS) as in New Zealand. It also uses the same Austroads guidelines. In addition, it included a large sample size. However, the study focused on pedestrian crashes and did not look into the safety of vehicular crashes, nor did it consider the operational performance of signalised intersections and this is considered a limitation of the study. In addition, it did not include pedestrian or traffic volumes.

However, the authors intended to develop an exposure based crash rate for each site, but it was considered impractical due to the large sample size (6,978 approaches). Also, multiyear pedestrian and traffic volume data were not available.

Zegeer et al. (2002) have established a relationship between the geometry of channelised right-turn lanes which permit turns at higher speeds and other unchannelised situations. Higher motor-vehicle speeds represent a higher risk to pedestrians crossing the roadway.

Therefore, in the event of a collision, vehicle speed directly affects the likelihood that a pedestrian will be fatally injured. For a pedestrian hit by a vehicle traveling at 32 km/h (20mph), the chance of being killed is 5%. For a 48-km/h (30mph) vehicle, that likelihood of a fatality rises to 45%, while for vehicles traveling at 64 km/h (40 mph), the likelihood of a fatal injury is 85%. Motorists traveling at higher speeds have less time to see pedestrians and require more time to slow, stop, or change direction to avoid striking them.

Schroeder et al. (2006) conducted a paired comparison study of blind and sighted pedestrians at three channelised right-turn locations. At each location, pedestrians were observed as they assessed gaps in traffic and identified opportunities to cross the channelised right-turn roadway.

Study participants consisted of nine visually impaired and nine sighted pedestrians, who were tested in pairs. They were asked to stand by the kerbside as though they were going to cross and indicate when they believed that it was safe to cross and when it was not safe. No actual crossings were performed in this experiment.

The key objective of that study was to identify whether the geometry of channelised right-turn lanes (CRTL) and/or the lack of signal control at channelised right-turn roadways negatively affect the delay and the safety for visually impaired pedestrians.

The findings show that crossings at all CRTL crossing locations are significantly more difficult to negotiate for visually impaired pedestrians than for sighted pedestrians. Blind pedestrians tend to face a greater risk and a greater amount of delay.

Furthermore, the research shows that conflicting traffic flow in the turning lane has a significant effect on crossing performance for both pedestrian groups.

The researchers noted that the study was limited to only two channelised right turn lane sites including three crosswalk locations, in the same geographic region. It was conceded that the performance of blind and sighted pedestrian populations may differ elsewhere.

Schroeder et al. (2011) carried out an observational study to assess the crossing behaviour of pedestrians with vision impairment over a channelised right-turn lane, and evaluated treatments to assist those pedestrians in undertaking the crossing. The study site was located in Charlotte, North Carolina. It consisted of four channelised right-turn lanes. The data collection was limited to two right turn lanes as shown in Figure 2-7. Sound strips with and without pedestrian actuated beacons (Flashers) were installed as safety treatments, as indicated in the Figure 2-7.

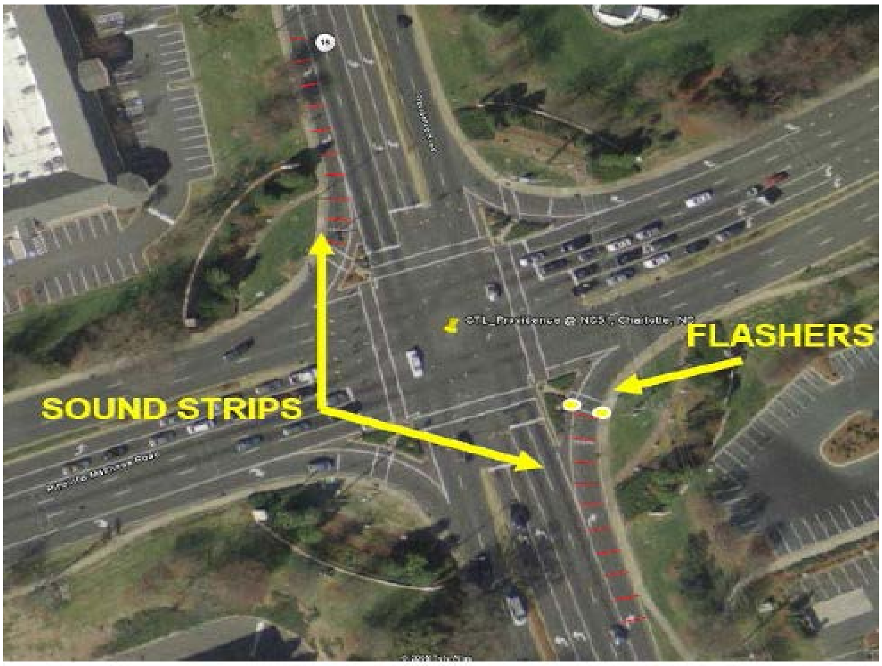


Figure 2-7 Study location of channelised right turn lane treatment (Schroeder et al., 2011)

In addition, pedestrians with vision impairment participated in both the before-treatment and after-treatment stages of the study. In the event that a participant moved into an unsafe situation, research team members were available to intervene. Analysis of the before and after treatment data collection recommended the following:

Before Treatment:

- ❖ The channelised right-turn lane study site experienced high traffic volumes;
- ❖ Pedestrian delays were relatively high;
- ❖ Gap acceptance and yield utilisation were relatively low; and
- ❖ Interventions by the research team members were in the range of 8% to 10%.

After Treatment:

- ❖ Yield rates of drivers increased slightly (from 15.2% to 22%) where sound strips were installed in combination with flashing beacons. There were no changes where sound strips were installed as the only treatment;
- ❖ Installation of treatments reduced, but did not eliminate interventions; and
- ❖ Several participants noted that they could hear better while making crossing decisions from the kerb than from the island. They stated that the sound of traffic behind them, when waiting on the island, made the crossing decision more difficult.

Potts et al. (2011) conducted observational field studies at 35 channelised right turn lanes (19 yield, 2 stop, 5 signals and 9 non-control). Over 2,800 pedestrian crossing observations were recorded at intersections designed with channelised right-turn lanes and pedestrian crossings. This study included observational field studies of pedestrian crossing behaviour and interviews with orientation and mobility (O&M) specialists who teach pedestrians with vision impairment to traverse intersections with channelised right-turn lanes.

The key results of the analysis were:

- Overall analysis showed that pedestrians do not have difficulty crossing channelised right-turn lanes (over 96% of the sample set). Avoidance manoeuvres, either by a pedestrian or a motorist, were observed in less than 1% of the pedestrian crossings;
- O&M specialists do not have a unified preference for crosswalk location at channelised right-turn lanes, but would like to see increased consistency in crosswalk locations. This would make it easier to teach pedestrians with vision impairment to traverse a channelised right-turn lane. In addition, they have a strong preference for raised islands with "cut-through" pedestrian paths, which provide better guidance for pedestrians with vision impairment than painted islands (note: painted islands are not allowed at signalised intersections in New Zealand);
- Use of a consistent design with respect to traffic control and crosswalk location is recommended; and
- Channelised right-turn lanes with acceleration lanes are very difficult for pedestrians with vision impairment to cross due to higher vehicle speeds and lower yield rates by motorists.

2.3.3 Safety of vehicle and pedestrian

A number of studies focused on the safety performance of both motor vehicles and pedestrians for different left turn configurations.

Potts et al. (2011) performed a cross-sectional safety analysis to evaluate the safety performance of intersection approaches with channelised right-turn lanes versus intersection approaches with other right-turn treatments. The study conducted on the signalised intersections was based on the following:

Seven years of motor-vehicle and pedestrian crash and volume data were obtained for 103 four-leg signalised intersections in Toronto, Ontario, Canada. Crash data for nearly 400 intersection approaches, including intersection approaches with channelised right-turn lanes (217 approaches), exclusive right-turn lanes (95 approaches), and shared through/right-turn lanes (83 approaches), were analysed to compare their safety performance.

The primary findings were:

- ❖ Channelised right-turn lanes had a lower crash rate (0.072 per year per approach) than exclusive right turn lanes (0.093), but a higher crash rate than shared through and right-turn lanes (0.051);

- ❖ Intersection approaches with channelised right-turn lanes appear to have similar motor vehicle safety performance as approaches with exclusive right-turn lanes or shared through/right-turn lanes. This was found to be the case at both the downstream end of the channelised right-turn lane (where the right-turning vehicle merges with the cross street) as well as at the upstream end of the channelised right-turn lane (where the right turning vehicle begins the right-turn manoeuvre);

- ❖ Intersection approaches with channelised right-turn lanes appear to have similar pedestrian safety performance as approaches with shared through/right-turn lanes. Intersection approaches with exclusive right-turn lanes have substantially more pedestrian crashes (approximately 70% to 80%) than approaches with channelised right-turn lanes or shared/through right-turn lanes. These results are opposite to those found by O'Brian et al. (2010) in their study; and

- ❖ The overall results of the safety analysis suggest that the three right-turn treatments may differ in motor-vehicle safety performance as the right turning vehicle merges with the cross-street vehicle (the downstream end of the channelised right-turn lane), but this was not conclusively established.

Al-Kaisy and Roefaro (2010) investigated the current state of practice regarding the use of channelised right-turn lanes (CRTLs) at signalised intersections. This includes current procedures and guidelines, type of traffic control, and the safety and operational experience of highway and local agencies in the United States.

The practice survey revealed the overall lack of knowledge about the operational and safety aspects of channelised right-turn lanes explaining, to a large extent, the lack of guidance in practice.

Survey results suggested that there is heavy reliance on the judgment of the highway agencies engineers in the use of channelised right-turn lanes and the selection of traffic control. Further, results confirmed a general perception in practice about the safety benefits of signal control at channelised right-turn lanes, despite the fact that such benefits were not supported by studies or statistics.

Survey participants were asked to evaluate their agency's safety experience with the use CRTL at signalised intersections for all control types; in general, and in particular the two most-common traffic controls used: yield control and signal control.

The summary of responses for state and local agencies is shown in Figure 2-8 and Figure 3-9 respectively.

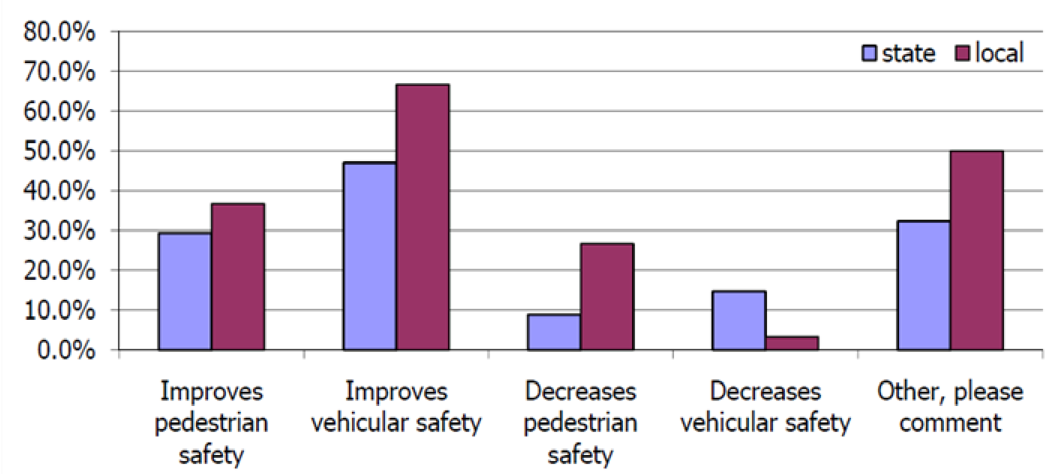


Figure 2-8 Safety experience with CRTL at signalised intersections (Al-Kaisy and Roefaro, 2010)

Approximately 49% of state agencies and 67% of local agencies believed that this treatment improves vehicular safety. It is obvious that the majority of highway agencies perceive CRTL at signalised intersections to improve vehicular safety. On the other hand, much lower percentages were reported for state and local agencies who believe that this treatment decreases vehicular safety (15% and 3% respectively).

- ❖ Relative to vehicular safety, there is less agreement among agencies regarding pedestrian safety. Fewer agencies perceive that this treatment provided improvements (30% of state and 37% of local agencies). More agencies thought of this treatment as decreasing pedestrian safety (9% of state and 27% of local agencies).

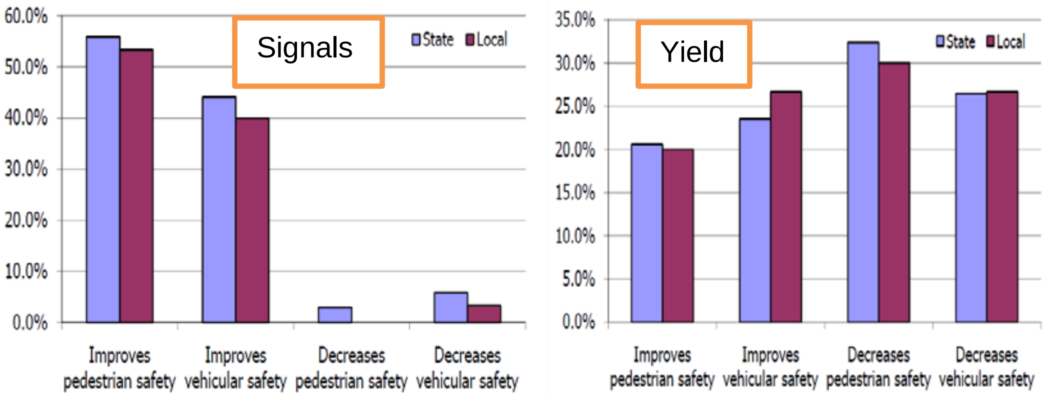


Figure 2-9 Safety experience with the use of signals and yield signs at CRTL (Al-Kaisy and Roefaro, 2010)

The key important findings of the practice survey at CRTL are summarised as follows:

- ❖ The decision on using CRTL and the type of traffic control heavily relies on engineering judgment by most state agencies. This is somewhat expected given the limited guidance available in national design documents and standards.

- ❖ The lack of guidance is particularly true for the selection of traffic control, as only 12% of state and 27% of local agencies reported the use of warrant studies in installing signal control at channelised right-turn lanes;
- ❖ There is an overwhelming perception by most state and local agencies about the safety benefits of signal control at channelised right-turn lanes. This conception is not supported by studies or statistics; and
- ❖ Vehicular traffic operation was the most prevalent consideration for using the CRTL and for the selection of traffic control.

It is worth noting that responses to the survey were based in most cases on personal observations, experiences, opinions, and perceptions. The lack of relevant data or studies was mentioned explicitly several times in the comments provided by survey respondents.

Although there was overwhelming agreement among participants about the operational benefits of channelised right-turn lanes, numerous comments were made about the lack of safety data or existing studies to answer this question. This emphasises the need for future research into the safety and operational aspects of left turn treatments at signalised intersections.

Turner et al. (2012) quantified the effects of signal phasing on various crash types for various travel modes at traffic signals, taking into account speed limits, intersection geometry and the land use environment. The data utilised represents a large number of variables using information collected for 238 low and high speed intersections from five cities throughout New Zealand and Melbourne, Australia.

Crash prediction models were developed for the predominant vehicle-vehicle crash types (right angle, right turn against, loss of control and rear end) and pedestrian-vehicle crash types (right angle, right turning). Vehicle crash models were developed for all-day and peak-time periods. It is worth noting that these models incorporated both pedestrian and traffic volumes.

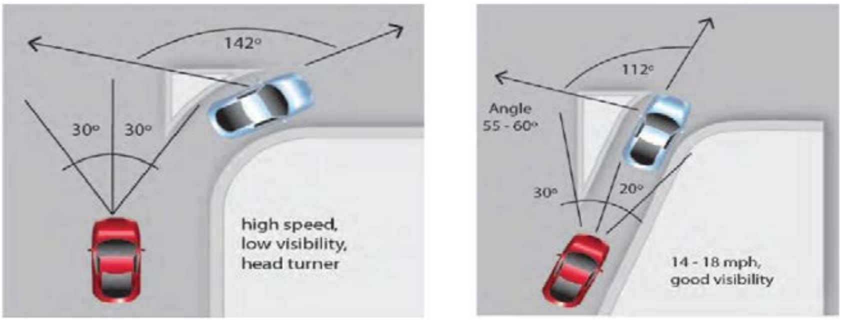
Two key related parameters were tested during the development of the crash prediction models: the presence of a free left turn for motor vehicles and combined shared lanes (shared through/right and through/left).

The results showed that the shared lanes increased right angle type crashes for motor vehicles and pedestrians. In addition, the presence of free left turns for motor vehicles increased the risk of loss of control, rear end and other crashes, with no effect on pedestrian crashes.

PedBikeInfo (2015) included a few notes related to the alignment of channelised right turn lanes and the angle between the channelised right-turn roadway and the cross street. These are divided into two types as shown in Figure 2-10:

- ❖ A flat-angle entry to the cross street (island shaped like an equilateral triangle, often with one curved side). This design is appropriate in channelised right-turn lanes with either yield control or no control, such as locations with an acceleration lane for vehicles at the entry to the cross street; and

- ❖ A nearly right-angle entry to the cross street (island shaped like an isosceles triangle). The nearly right-angle entry design can be used with stop sign control or traffic signal control for vehicles at the entry to the cross street; yield control can be used with this design where the angle of entry and sight distance along the cross street are appropriate.



Channelized Right-Turn Lane With Flat-Angle Entry to Cross

Channelized Right-Turn Lane With Nearly Right-Angle Entry to Cross Street

Figure 2-10 Typical channelised right turn lane with different entry angle (PedBikeInfo, 2015)

To improve both vehicle and pedestrian safety at CRTL, the right-angle entry (High Entry Angle in New Zealand) should be adopted into the design. The right entry angle of the CRTL improves safety by:

- ❖ Slowing vehicle turns while still allowing for large vehicles;
- ❖ Allowing pedestrian crossings to be placed into the driver's view during the approach; and
- ❖ Allowing for the use of raised crossings such as zebra on raised platform.

It also noted that visually impaired pedestrians have concerns with using CRTL including:

- ❖ Difficulty in sensing where the crosswalk is located; and
- ❖ Difficulty in sensing when vehicles have yielded the right-of-way.

2.3.4 Safety of cyclist

A few researchers investigated the safety elements and type of crashes involving cyclists at signalised intersections, in particular, whether the respective crashes occurred at certain types of left turn facility.

Hunter et al. (2002) conducted a before and after study in the conflict zone where the paths of bicyclists and motorists crossed. This conflict zone was treated with blue markings combined with yield signs for motorists at 10 signalised intersections in Portland, Oregon. The study involved video observations and feedback from approximately 850 bicyclists and 190 motorists in the before phase, and 1,020 bicyclists and 300 motorists in the after phase.

The key results were as follows:

- There was a significant increase in motorists yielding to bicyclists after the treatment was installed - from 71% in the before period to 87% in the after period;
- Significantly more bicyclists followed the path marked for bicyclists after the blue markings were in place – from 85% in the before period compared to 93% in the after period;
- There was a decrease in head-turning and scanning on the part of bicyclists after the treatment was installed - from 43% in the before period to 26% in the after period, which was a concern. The authors were not sure about the reason for this result;
- While conflicts between the two modes were rare, the conflict rate decreased from 0.95 conflicts per 100 entering bicyclists in the before period to 0.59 conflicts per 100 entering bicyclists in the after period; and
- The bicyclists surveyed thought the treatment would increase safety by 76%.

Austroads research (2011) included ‘before and after’ cross-sectional analyses for a range of cycle facilities in New Zealand (Christchurch) and Australian cities (Adelaide). This study included a total of 383 approaches at 102 signalised crossroads. One of the key crash types was a left turn side-swipe (left turn vehicle cutting off straight through cyclists). The key conclusions from the ‘before and after’ and cross-sectional analyses that related to the left turn types are:

- Sites with shared left-turn and through lanes have higher initial crash rates; and
- Sites with exclusive left turn lanes (including left turn slip lane) are much safer for cyclists than those with a shared through and left turning lane. Any cycle lanes provided the need to use colour from the transition across the diverge area to the limit line.

2.3.5 Safety in general

A number of studies reviewed assessed in general the safety of left turn lanes at signalised intersections without specific focus on any of the modes of transport.

Ogden et al. (1994) undertook a qualitative assessment of factors, other than traffic volumes, affecting crash patterns at signalised intersections (accident prediction model for signalised intersections in Melbourne). The study identified intersections performing as expected, and better or worse than expected labelled: “normal”, “low” and “high”, respectively.

The classified intersections were quantitatively assessed using number of lanes, presence of shared or exclusive right turn lanes, slip lanes, exclusive left turn lanes (no slip lane), lane width, median presence and width, tram/bus stop presence, signal mast arms, gradient, right turn control, clearway presence, surrounding land use (industrial, commercial, educational, residential or other), and presence of service roads.

Figure 2-11 shows the distribution of left turn configuration (left slip versus left no slip) for the three groups – low, normal and high. The results show that left turns featuring a slip lane appear to be safer. However, the percentages are quite small (76 sites included but the number of approaches were not mentioned) and this result is not considered to be conclusive.

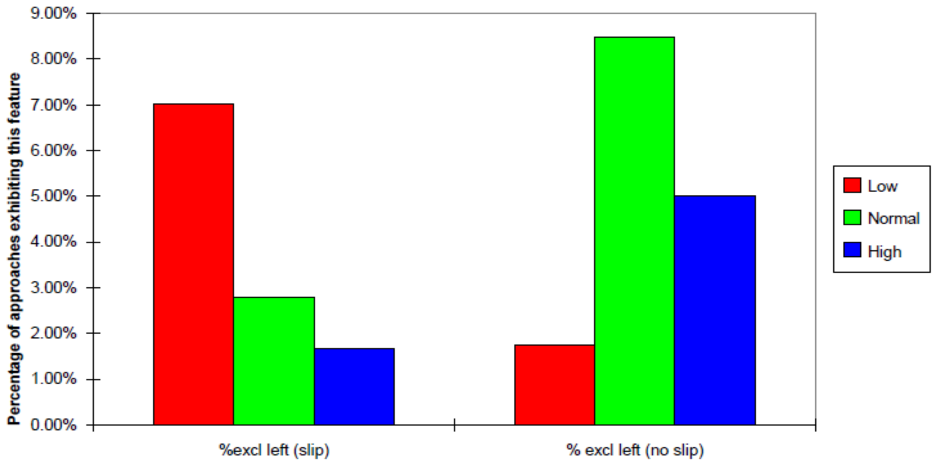


Figure 2-11 Left turn configuration by intersection group (Ogden et al., 1994)

Staplin et al. (1997) conducted an accident analysis in the US to examine the problems facing elderly drivers at intersections. Approximately 700 accident records were reviewed during this analysis. In general, it was found that older drivers had difficulty yielding the right-of-way and making right turns at intersections, but the accident analysis did not reveal channelised right turns as a safety issue.

Tarawneh and McCoy (1996) conducted a field investigation in the US to study the effects of the geometrics of right-turn lanes on the turning performance of drivers. Right-turn performance of 100 subjects (three age groups) was evaluated at four signalised intersections of different right-turn lane channelisation and skew. Three of the four intersections had a channelised right-turn lane. The channelised right-turn lanes were controlled by yield signs.

The investigation found that drivers turn right at speeds of 5 to 8 km/h higher on intersection approaches with channelised right turn lanes than they do on approaches with unchannelised right-turn lanes. In addition, it was observed that drivers are less likely to come to a complete stop before turning onto the cross street.

Abdel-Aty and Nawathe (2006) used an artificial network to analyse the safety of signalised intersections. Geometry, traffic, and crash data were obtained for 1,562 signalised intersections in Florida. Neural network trees were used to determine the relationship between intersection geometry/configuration and the frequency of specific types of crashes. The study found that:

- The presence of channelised right-turn lanes on major roads had no significant effect on total crashes, but was linked to an increase in turning and sideswipe crashes; and
- On the minor roads, the presence of channelised right-turn lanes was associated with a decrease in the total crashes and an increase in rear-end crashes.

Harwood et al. (2002) conducted a before and after evaluation of the safety effects of providing left turn and right turn lanes for a total of 280 improved intersections (in rural and urban areas). 100 out of the 280 intersections in the study were urban and signalised. The types of improvement projects evaluated included those installing additional left-turn and right-turn lanes, and extensions to the length of existing left or right-turn lanes. The research developed quantitative safety measures for this type of installation at left turn treatments.

The study used observational data acquired before and after safety evaluations. It concluded the following two key results:

- Additional right-turn lanes are effective in improving safety at urban signalised intersections. Installation of a single right-turn lane on a major-road approach would be expected to reduce total intersection accidents by 4 percent at urban signalised intersections; and
- Additional right-turn lane installation reduced accidents on individual approaches at four-leg intersections by 18 percent in urban areas.

Austroads (2015) analysed crash data over a five-year study period (2006–10). A series of site investigations was also conducted at ‘high’ crash sites in New South Wales, Victoria and Queensland to identify factors that may have contributed to the occurrence or severity of rear-end crashes. One of these factors was the provision of left turn configuration and the type of traffic control. The key results found:

- Exclusive left-turn lanes were less common. Ten out of the 25 (40%) intersection approaches, where left-turn traffic was apparent, did not feature a dedicated left-turn lane. A further 14 (56%) featured one left-turn lane only. Therefore, it is possible that introducing or adding an additional left-turn lane may be effective in reducing the rear-end crash rate; and
- Ten of the 25 (40%) left-turn approaches featured combined left-turn/through lanes. At these approaches the through traffic would have a different travel speed to left-turning traffic, which needs to decelerate in order to safely conduct the turn. This difference in travel speeds would

increase the rear-end crash risk. There may be a benefit in separating these traffic streams.

Baldock et al. (2005) conducted a crash analysis for a 5-year period and indicated that slip lanes allow left-turning traffic to dissipate faster, thus reducing traffic queues that contribute to rear-end crashes (as a countermeasure to reduce rear-end crashes).

Wang and Abdel-Aty (2006) also analysed the rear-end crashes at signalised intersections and found that slip lanes and exclusive left-turn lanes treatments could reduce the incidence of rear-end crashes by 69% and 31% respectively.

Kumara and Chin (2005) developed a mathematical model that correlates accident frequencies to causal factors at three legged signalised intersections in Singapore. A total of 104 three-legged signalised intersections were selected for the model development. These represent about 40% of such intersections in all of Singapore. Each intersection was divided into separate approaches, and accident and other data were taken at each approach.

The model showed that the uncontrolled left-turn slip, which allows left-turning vehicles to merge into the cross-traffic stream, increases the likelihood of accidents by approximately 13%. However, by providing an acceleration section in the left-turn lane, drivers may be able to merge more easily. This explains a reduction in accidents of about 37% when such lanes were provided, and everything else was constant. However, the type of the slip lane angle was not included.

2.3.6 Summary of safety performance

Numerous studies on the safety effects on various road users-motor vehicles, pedestrians, and cyclists- for different left turn treatments have been reviewed. There was only one safety study reviewed that included all road users conducted by Potts et al. On the other hand, most of the studies focussed on a particular type of road user. The primary results of these studies are summarised in Table 2-5.

Table 2-5 Summary of key finding on the safety performance

Studies Reviewed	Summary of key findings on safety for different left turn lane Treatments
Vehicle Safety	
Dixon et al. (2000) and Fitzpatrick et al. (2006)	The slip lane had the highest number of crashes in the Dixon Study and the second highest number of crashes in the Fitzpatrick study. In both studies, the shared lane had the lowest number of crashes. The number of sites was limited in both studies.
Pedestrian Safety	
O'Brien et al.	Slip lane crashes and injuries occurred in a much

(2010)	smaller proportion than their frequency on the network. Therefore, it may offer safety advantages to pedestrians and should continue to be considered as a default left-turn treatment at new intersections; and the second safer treatment is an exclusive left turn lane. Shared lanes appeared to perform poorly and these experienced significantly more crashes and injuries.
Zegeer et al. (2002)	Higher motor-vehicle speeds represent a higher risk to pedestrians crossing the slip lanes designed with high kerb radius.
Schroeder et al. (2006)	Crossings at slip lanes are significantly more difficult to negotiate for visually impaired pedestrians than for sighted pedestrians. Visually impaired pedestrians tend to face a greater risk and a greater amount of delay. Only two locations were studied.
Schroeder et al. (2011)	Give-way rates of drivers to pedestrians increased at slip lanes where sound strips and flashing beacons were installed.
Vehicle and Pedestrian Safety	
Potts et al. (2011)	The left turn slip lanes had a lower vehicle crash rate than exclusive left turn lanes. The left turn slip lanes had similar pedestrian safety performance as the shared through/ left turn lanes.
Al-Kaisy and Roefaro (2010)	The survey results showed that high percentages of state agencies and local agencies believed that left turn lanes improve vehicular safety. There is less agreement among agencies in regards to pedestrian safety.
Turner et al. (2012)	Shared lanes increased right angle type crashes (HA type) for motor vehicles and pedestrians. The presence of free left turns for motor vehicles increased the risk of loss of control, rear end and other crashes, with no effect on pedestrian crashes.
PedBikeInfo	The safety of left turn slip lanes could be improved by adopting the high entry angle into the design.
Cyclist Safety	
Hunter et al. (2002)	There was a significant increase in the of percentage motorists of giving-way to bicyclists after marking the conflict zone: increased from 71% to 87%.

Austroads research (2011)	Sites with exclusive left turn lanes (including left turn slip lane) are much safer for cyclists than those with a shared through and left turn lanes.
General Safety	
Ogden et al. (1994	The left turn slip lanes appear to be safer than other non-slip lanes.
Staplin et al. (1997)	It was observed that elderly drivers had difficulty giving-way at left turn slip lanes. But the accident analysis did not reveal this observation as a safety issue.
Tarawneh and McCoy (1996)	It was found that drivers were turning left at speeds 5 to 8 km/h higher on intersection approaches with left turn slip lanes than they were on approaches with non-slip lanes. Limited samples.
Abdel-Aty and Nawathe (2006)	The presence of left turn slip lanes on major roads had no significant effect on the total number of crashes. On the other hand, for the minor roads, it decreased the total number of crashes, as well as the number of rear-end crashes.
Harwood et al. (2002)	Installation of a left turn lane reduced accidents on individual approaches by 18% in signalised intersections.
Austroads (2015)	It was found that exclusive left-turn lanes were safer and may be effective in reducing the rear-end crash rate more than the combined left-turn/through lanes.
Bauer and Harwood (1996)	The left turn lane slip lanes resulted in an increase in total multiple-vehicle crashes and injury crashes.
Baldock et al. (2005)	It is claimed that left turn slip lanes allow left-turning traffic to dissipate faster, thus reducing traffic queues that contribute to rear-end crashes.
Wang and Abdel-Aty (2006)	It was found that left turn slip lanes and exclusive left-turn lanes treatments could reduce the incidence of rear-end crashes by 69% and 31% respectively.
Kumara and Chin (2005)	Providing an acceleration lane at uncontrolled left-turn slip lanes may reduce accidents by 37%.

2.4 Operational performance

Chandler et al. (2013) indicated that the left turn slip lane is expected to reduce delays for the through movement as through vehicles do not have to slow down for decelerating vehicles negotiating a right turn. In addition, right-turning vehicles are likely to be forced to queue behind through vehicles when provided a right-turn slip lane, resulting in a reduction in the control delay. Therefore, it may increase the intersection capacity by re-allocating more green time to other movements.

Potts et al. (2011) conducted an extensive traffic operational analysis to evaluate the operational performance of right-turning vehicle movements at signalised intersections for three configurations: an exclusive right-turn lane, a yield-controlled channelised right-turn lane, and a signalised channelised right turn lane. A series of micro-simulation runs using VISSIM were conducted to evaluate the traffic operational performance for both vehicles and pedestrians.

This is the only study found that investigated the operational performance for differing left turn treatments.

The overall results of the simulation modelling showed that channelised right-turn lanes can substantially reduce delay for right-turning vehicles in nearly every traffic volume scenario. Site specific factors, such as pedestrian volumes and the geometry of the channelised right-turn lane, have an effect on the level of improvement. Therefore, these factors are important in determining the delay benefits that may result from installation of the channelised right-turn lane.

The following were the key findings:

- ❖ Channelised right-turn lanes with yield control were shown to reduce right-turn delay to vehicles by 25% to 75% in comparison to intersection approaches with exclusive right-turn lanes. High pedestrian volumes increase right-turn delay by approximately 60% on a yield-controlled channelised right-turn lane;
- ❖ The addition of an acceleration lane at the downstream end of a channelised right-turn lane can substantially reduce the right-turn delay by 65% to 85%, depending on the conflicting traffic volume on the cross street. However, channelised right-turn lanes with acceleration lanes are very difficult for pedestrians with vision impairment to cross due to vehicle speeds and lack of yielding by motorists; and
- ❖ For channelised right-turn lanes with signal control. The use of an overlap phase, or other method of providing additional green time to right-turning vehicles, can substantially reduce the delay for these movements.

2.5 Implications of literature findings

The main key findings of the literature review may be summarised as follows:

- There is a lack of combined research in the area of safety and efficiency of left turn treatments both in New Zealand and overseas. This confirms the need for a research to be initiated in this area.
- There are limited research studies on the operational effects of different left turn treatments. No studies were found in New Zealand and Australia; however, there was one study found in the US that examined the operational performance of differing left turn treatments;
- The analysis of safety performance of the different left turn treatments for the majority of studies were based on approach level analysis rather than the intersection level analysis. Consequently, the study methodology was adjusted to reflect this;
- Two comprehensive studies stood out from this literature review: O'Brian et al. (2010) and Potts et al. (2011).These two studies will be used as a comparison in this study analysis in conjunction with other reviewed studies.
- Several studies were found on the safety aspects of different left turn treatments on road users: motor vehicles, pedestrians and cyclists each tackled independently. On the other hand, there was only one study conducted by Potts et al. (2011) on the safety performance on all road users, and including the operational aspects of different left turn treatments;
- A number of guidelines have been reviewed, mainly in New Zeeland and Australia, for context. These guides detail the design elements of different left turn treatments. There was little or no research done on the safety and the operational benefits/disbenefits of any particular type of left turn treatments. This supports the need for a study to investigate these aspects;
- A number of studies recommended the need for further research on the operational and safety aspects of the left turn treatments at signalised intersections;
- Some key questions related to the traffic operation of left turn slip lanes need to be investigated include:
  - What is the operational performance of left turn slip lanes?
  - What are the operational benefits/disbenefits that would be gained / lost if the left turn slip lane were removed?
  - What is the operational performance of other left turn treatments compared to the left turn slip lane?

3 Data Collection and Methodology

This chapter outlines the main data collection procedures and the methodological approach used in the study. Furthermore, it provides guidance and recommendations to researchers conducting additional research in this area.

The data collection for this research was a challenging and time consuming task. It took approximately a year to prepare both the intersection and the crash data sets for analysis. This is due to the limitations of the data availability in the RCA systems. As a result, extensive manual interrogation was required to gather such data.

3.1 Study area

The study area is the Auckland region as shown in Figure 3-1. Auckland is the largest city in New Zealand. Currently in the wider Auckland Region there are approximately 850 signalised sites, excluding ramp metering signals. The 850 signalised sites include midblock pedestrian crossings, intersections, interchanges, and roundabout metering signals.

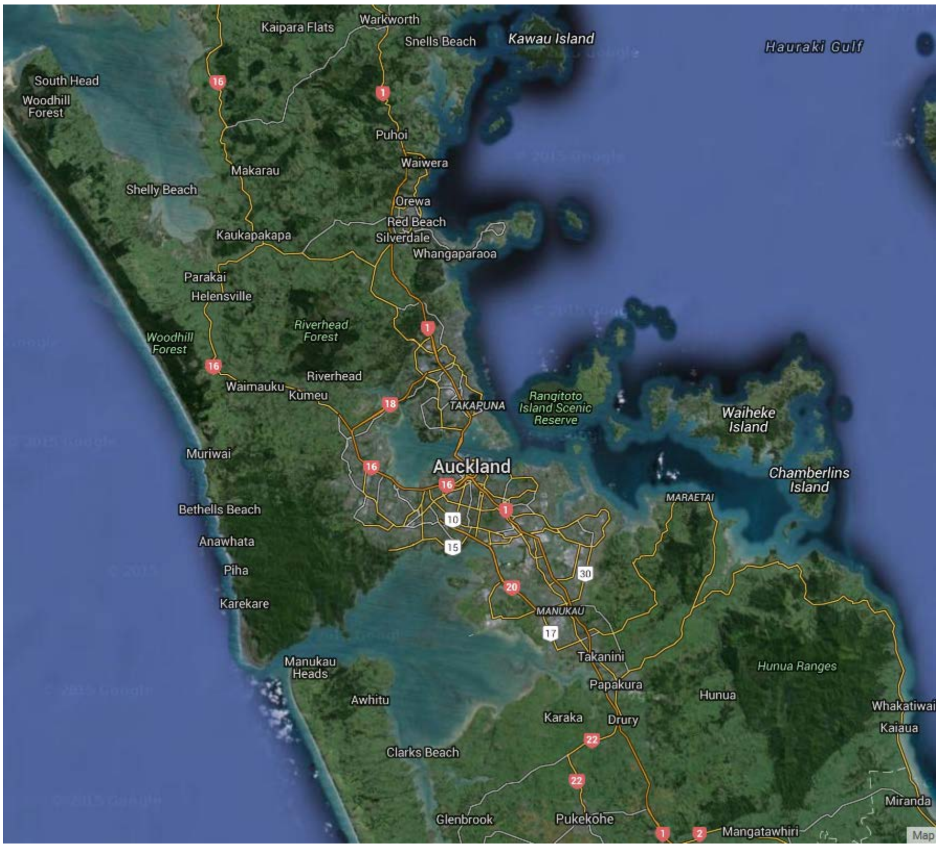


Figure 3-1 Study Area

The traffic signals network in Auckland is divided into four geographical areas: north, south, west and central. These areas are managed and operated by the Auckland Transport Operation Centre located at Smales Farm (ATOC- Smales). The list of signalised sites was obtained as raw data from RAMM, and then shape files were created and exported into a GIS environment and plotted on a map. An overview of the Auckland signals network is depicted in Figure 3-2.

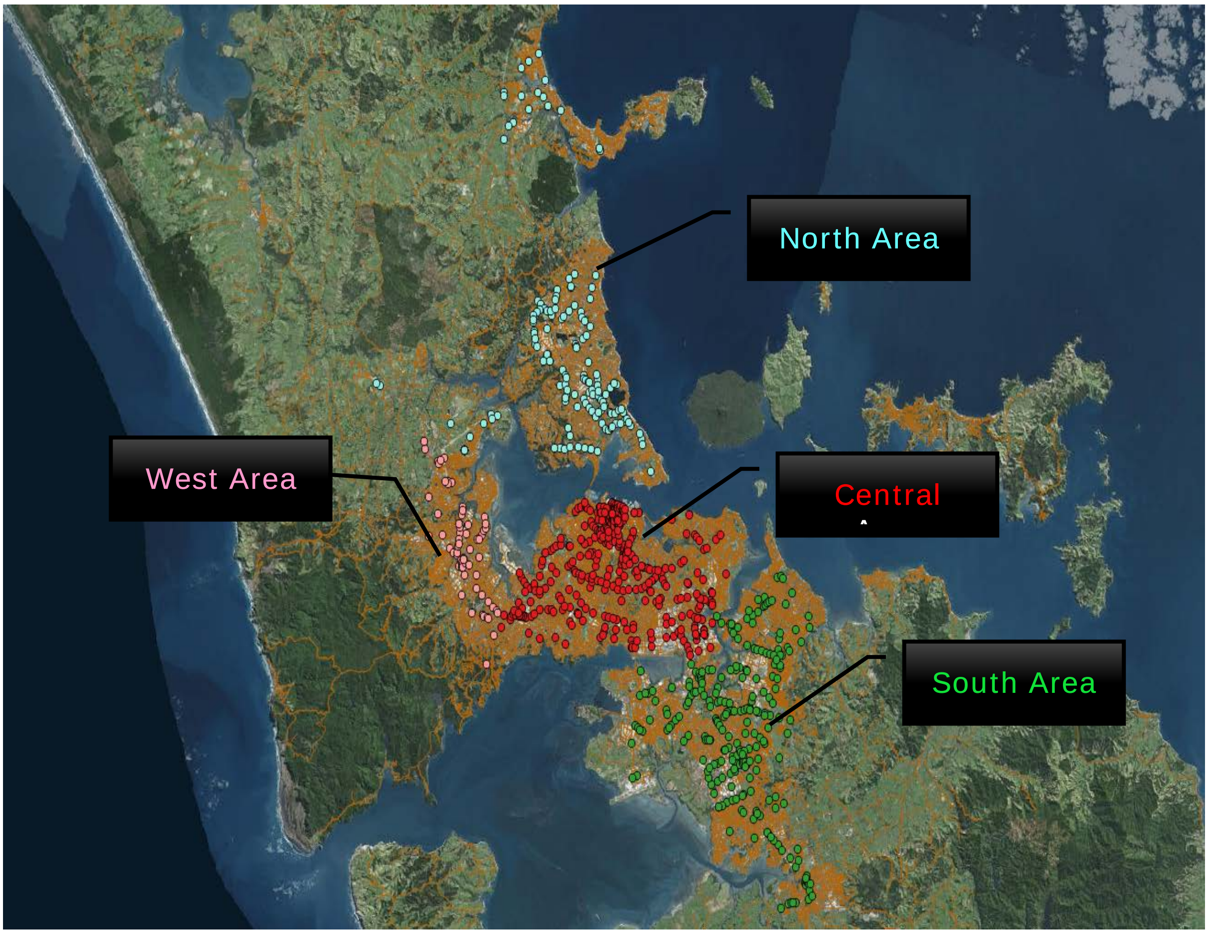


Figure 3-2 Traffic signals network in Auckland

3.2 Data collection procedures

For the purpose of this research, there were two main types of data collected from the study area:

- **Signalised intersection data:**
  - Splitting the whole signals network into intersections approaches
  - Categorising the intersections approaches into left turn treatment types
- **Crash data for each signalised site:**
  - Obtaining crash data for the whole signals network

- Identifying crashes involving left turn movements
- Assigning the crashes to each of the approaches

The data collection procedure and methodology are described in detail in the following section.

3.2.1 Intersection data collection and reduction

The list of all signalised sites in the Auckland region were obtained from the RAMM System in an Excel format. The list included the site name, site number (SCATS site identification number), installation date, and X and Y coordinates. There was initially a total of 848 signalised sites in the RAMM database as of March 2015 as well as the proposed signalised sites. After examining the list, sites with the following criteria were excluded:

- ❖ Sites installed in 2014 and 2015, as they do not have enough historic crash data;
- ❖ Proposed (planned) signalised intersections;
- ❖ Signalised midblock pedestrian crossings;
- ❖ Roundabout metering signals;
- ❖ Intersections on one-way streets; and
- ❖ Intersections not connected to SCATS.

The number of sites excluded from the initial list is presented in Table 3-1

Table 3-1 List of sites excluded from the initial list of the intersections

Site Exclusions	**Number of sites**
Midblock pedestrian crossings	154
Roundabout metering signals	4
New installations	22
Intersection on one-way streets	8
Proposed (planned) signalised intersections	33
Not connected to SCATS	2
Total number of sites	848
Total excluded sites	223
Remaining sites for the study	**625**

3.2.2 Left turn network classification

Each of the 625 signalised sites was categorised by approaches, whether it had 3, 4 or 5 approaches, depending on whether it was a tee junction, a cross roads intersection or a multi leg intersection. Then the data of each left turn treatment type was manually collected for each individual approach, for all signalised intersections. This data was not available from a data set, so it was rearranged and grouped as follows:

- Type of left turn lane: conventional lane or slip lane;
- Type of left turn control: free flow, signalised, or give-way; and
- Type of pedestrian facilities on left turn slip lane such as zebra crossing and/or zebra crossing on a raised table.

The data showing each left turn treatment type for each approach was manually retrieved using various sources including Google Earth, Street View Map, Auckland GIS Viewer, and verified by SCATS and controller information sheets (CIS). This left turn classification took a considerable time. A snapshot of the left turn classification spread sheet is shown in Figure 3-3.

		Intersection Details		Treatment			Control			Ped Facility				Treatment			Control			Ped Facility				Treatment			Control			Ped Facility				Treatment			Control			Ped Facility		
AREA	Number	Description	North Approach	Slip Lane	Exclusive	Shared	Giveway	Signalised	Free Flow	Zebra	Raised Zebra	Comment	South Approach	Slip Lane	Exclusive	Shared	Giveway	Signalised	Free Flow	Zebra	Raised Zebra	Comment	East Approach	Slip Lane	Exclusive	Shared	Giveway	Signalised	Free Flow	Zebra	Raised Zebra	Comment	West Approach	Slip Lane	Exclusive	Shared	Giveway	Signalised	Free Flow	Zebra	Raised Zebra	Comment
North	1101	WAIRAU RD - ARCHERS RD	Wiarau	1			1						WAIRAU RD			1																	ARCHERS RD		1							
North	1102	WAIRAU RD - TRISTRAM AVE - HILLSIDE RD	WAIRAU RD	1				1					WAIRAU RD			1							TRISTRAM AVE	1			1						HILLSIDE RD	1			1					
North	1103	WAIRAU RD - DIANA DR	WAIRAU RD			1																											Diana Dr	1			1					
North	1104	WAIRAU RD - TARGET RD											WAIRAU RD	1			1						Target	1				1														
North	1105	ARCHERS RD - POLAND RD - PAK 'N SAVE ENTRANCE	Poland Rd		1								Pak N Save		1								Archers Rd		1								Archers Rd		1							
North	1107	WAIRAU RD - PORANA RD											WAIRAU RD			1																	Porana Rd	1			1					
North	1108	TRISTAM AVE - FORREST HILL RD											Forrest Hill Rd	1						1													Tristam Ave	1				1				
North	1109	SUNNYNOOK RD - BECROFT DR	SUNNYNOOK RD	1						1													Becroft Dr		1																	
North	1110	TARGET RD - LINK DR	Target Dr			1																	Link Dr	1						1												
North	1201	ANZAC ST - AUBURN ST	Auburn St		1								Auburn St	1				1					Anzac St		1								Anzac St			1						
North	1202	ANZAC ST - BARRYS POINT RD - PUPUKE RD	Lake Pupuke Rd			1							Barrys Point Rd	1						1			Anzac St	1						1			Anzac St			1						
North	1203	TAHAROTO RD - DOMINION ST											Dominion St		1								Anzac St		1																	
North	1204	TAHAROTO RD - NORTHCOTE RD	Northcote Rd		1								Northcote Rd	1						1			Taharotro Rd	1						1			Taharotro Rd			1						
North	1205	TAHAROTO RD - SHAKESPEARE RD - WAIRAU RD	WAIRAU RD	1						1			TAHAROTO RD	1						1			SHAKESPEARE R	1						1			SHAKESPEARE RD	1						1		
North	1208	NORTHCOTE RD - SUNNYBRAE RD - AKORANGA DR	SUNNYBRAE RD	1						1			AKORANGA DR			1							Northcote Rd	1						1			Northcote Rd	1						1		
North	1209	WAIRAU RD - FORREST HILL RD	Wiaraub Rd			1																	Forrrest Hill rd	1																		
North	1210	AKORANGA DR - THE WAREHOUSE WAY																																								
North	1211	TAHAROTA RD - KILLARNEY ST - FRED THOMAS DR	KILLARNEY ST			1							FRED THOMAS DR		1								Taharoto Rd	1						1			Taharoto Rd	1						1		
North	1212	TAHAROTA RD - SMALES FARM ENTRANCE											TAHAROTO RD	1						1													SMALES FARM	1						1		
North	1213	AKORANGA DR - COLLEGE RD	Akoranga Dr			1							Akoranga Dr			1							College Rd			1							College Rd		1							
North	1215	WAIRAU RD - WESTLAKE G. H. S.											Wairau Rd			1																	WESTLAKE G. H. S			1						
North	1216	NORTHCOTE RD - SMALES FARM - TAKAPUNA INT. SCHOOL	The Avenue (Samle	1							1																						Northcote rd	1						1		
North	1217	SHAKESPEARE RD - NORTH SHORE HOSPITAL DRIVE	SHAKESPEARE RD		1								SHAKESPEARE RD		1								NORTH SHORE HOSPITAL DRIVE			1																
North	1301	LAKE RD - BAYSWATER AVE - WILLIAMSON AVE	Lake Rd			1							Lake Rd			1							Williamson Ave	1						1			Bayswater Ave	1						1		
North	1302	LAKE RD - BARDIA ST - WINSCOMBE ST	Lake Rd			1							Lake Rd			1							Winscombe St			1							Winscombe St			1						
North	1304	LAKE RD - JUTLAND RD - HAURAKI RD	Lake Rd	1						1			Lake Rd			1							Hauraki Rd			1							Jutland Rd		1							
North	1305	LAKE RD - ESMONDE RD											Lake Rd	1				1					Esmonde Rd		1																	
North	1306	LAKE RD - HURSTMERE RD - THE STRAND - NORTHCROFT ST	Lake Rd	1						1			Lake Rd			1							The Strand	1						1			Northcroft St			1						
North	1307	ESMONDE RD - BURNS AVE	Burns Ave			1																											Esmonde Rd	1					1			
North	1310	ESMONDE RD - ASSEMBLY OF GOD CHURCH ENTRANCE											God Church Entran	1			1						Esmonde Rd	1			1															
North	1401	WAIRAU RD - GLENFIELD RD											GLENFIELD RD	1						1			Wairau RD	1						1												
North	1402	GLENFIELD RD - MANUKU RD - HOGANS RD	GLENFIELD RD		1								GLENFIELD RD	1						1			Hogans Rd		1								Manuku Rd		1							
North	1403	GLENFIELD RD - BENTLEY AVE	GLENFIELD RD	1						1													Bentley Ave	1						1												
North	1404	GLENFIELD RD - DOWNING ST	Glenfield Rd	1			1																Downing St	1			1															
North	1405	GLENFIELD RD - ESKDALE RD											Glenfield Rd	1			1						Eskdale Rd	1			1															
North	1406	GLENFIELD RD - BIRKENHEAD AVE - PUPUKE RD	Glenfield Rd	1			1																Pupuke Rd		1																	
North	1407	GLENFIELD RD - KAIPATIKI RD	Glenfield Rd	1			1						Glenfield Rd			1							Westfield Mall	1			1						Kaipatiki Rd	1						1		
North	1408	MANUKU RD - EASTON PK PDE inc remote PED XING											Easton Park Pde		1								Manuka Rd		1																	
North	1409	GLENFIELD RD - ALBANY HWY - SUNSET RD - GLENDHU RD	Albany Hwy		1								Albany Hwy			1							Sunset Rd	1						1			Glendhu Rd	1						1		
North	1413	GLENFIELD RD - CHIVALRY RD - MAYFIELD RD	GLENFIELD RD	1						1			GLENFIELD RD			1							Chivalry Rd	1						1			Mayfield Rd			1						
North	1502	ONEWA RD - QUEEN ST											Queen St	1				1															Onewa Rd			1						
North	1503	ONEWA RD - LAKE RD	Lake Rd		1																												Onewa Rd			1						
North	1506	ONEWA RD - BIRKENHEAD AVE - HIGHBURY BYPASS	BIRKENHEAD AVE	1						1			BIRKENHEAD AVE	1						1			Onewa Rd	1						1			Highbury Bypass	1						1		
North	1507	MOKOIA RD - CHELSEA VIEW DR											Chelsea View Dr			1							Mokoia Rd			1																
North	1508	MOKOIA RD - COLONIAL RD - HIGHBURY BYPASS											Mokoia Rd		1								HIGHBURY BYPAS	1						1												
North	1601	KITCHENER RD - MILFORD RD - DODSON AVE																																								
North	1602	KITCHENER RD - SHAKESPEARE RD - OMANA RD																																								
North	1603	EAST COAST RD - HASTINGS RD	EAST COAST RD	1						1			EAST COAST RD			1							Hastings Rd		1																	

Figure 3-3 Snapshot of left turn classifications

3.3 Intersection data analysis

3.3.1 Typical left turn treatments at signalised intersections

There are seven left turn treatments typically used at signalised intersections, as follows:

- ❖ Conventional exclusive lane;
- ❖ Conventional shared lane (shared through and left or shared left and right);
- ❖ Slip signal control lane, with signalised pedestrian crossing marking;
- ❖ Slip give-way control lane, with zebra pedestrian crossing marking;
- ❖ Slip give-way lane control, with no pedestrian crossing marking;
- ❖ Slip lane with give-way control, with zebra pedestrian crossing marking on raised table; and
- ❖ Slip lane free flow (no control).

These seven left turn treatments and their associated control are shown in Figure 3-4 to Figure 3-10.

Figure 3-4 Exclusive left turn lane

Figure 3-5 Shared left turn lane

Figure 3-6 Slip lane with signalised pedestrian crossing

Figure 3-7 Slip lane with zebra pedestrian crossing

Figure 3-8 Slip lane give-way with no pedestrian crossing

Figure 3-9 Slip lane with zebra pedestrian crossing on raised table

Figure 3-10 Slip lane free flow

3.3.2 Summary of left turn network classification

A summary of left-turn treatments at signalised intersections in Auckland is presented in Table 3-2.

Table 3-2 Summary of left turn treatments at signalised intersections in the Auckland region

Left turn treatment type	Number and proportion of the treatments in the signals network	
	Number	Percent
Left Turn Slip Lane		
Signalised Control	74	4%
Free Flow	43	2%
Give-way No Pedestrian Marking	463	25%
Zebra Crossing	160	9%
Raised Zebra Crossing	1	0%
Slip Lane Total	**741**	**41%**
Conventional Left Turn Lane		
Exclusive Lane	389	21%
Shared lane	688	38%
Conventional Lane Total	**1077**	**59%**
Total no of LT approaches	**1818**	**100%**

Examining the remaining 625 signalised intersections, resulted in 1818 approaches that allow left turns.

Figure 3-11 depicts the distribution of the total left turn treatment type.

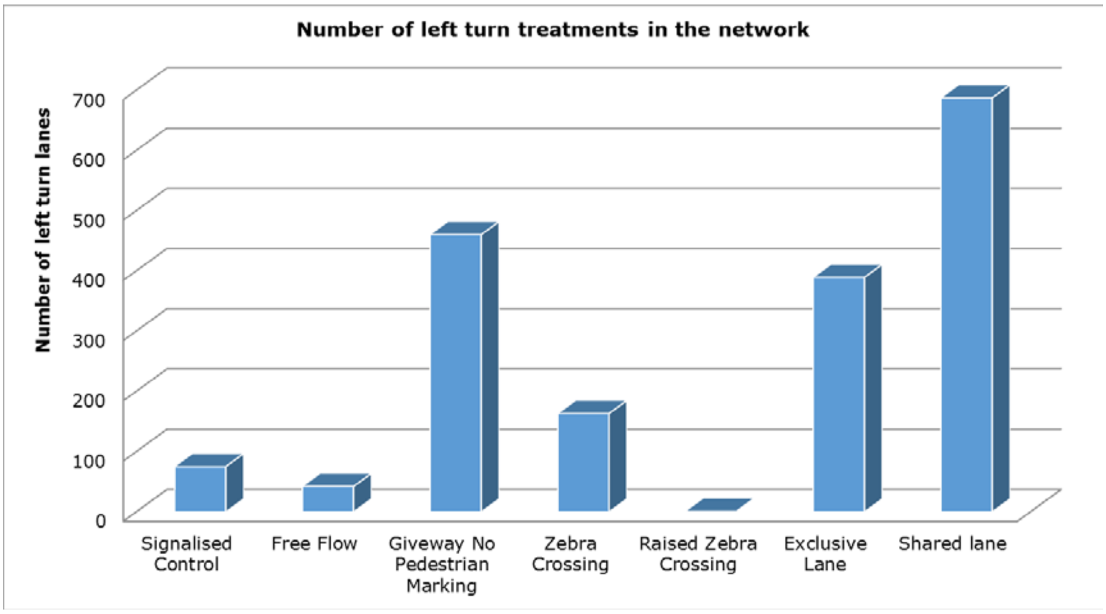


Figure 3-11 Distribution of the number of left turn treatments in the signals network

Figure 3-12 and Figure 3-12 show the distribution of the slip lane and the conventional left turn types. The graphs show that the highest left turn types in the signal network are slip lane with give-way control and conventional shared lane.

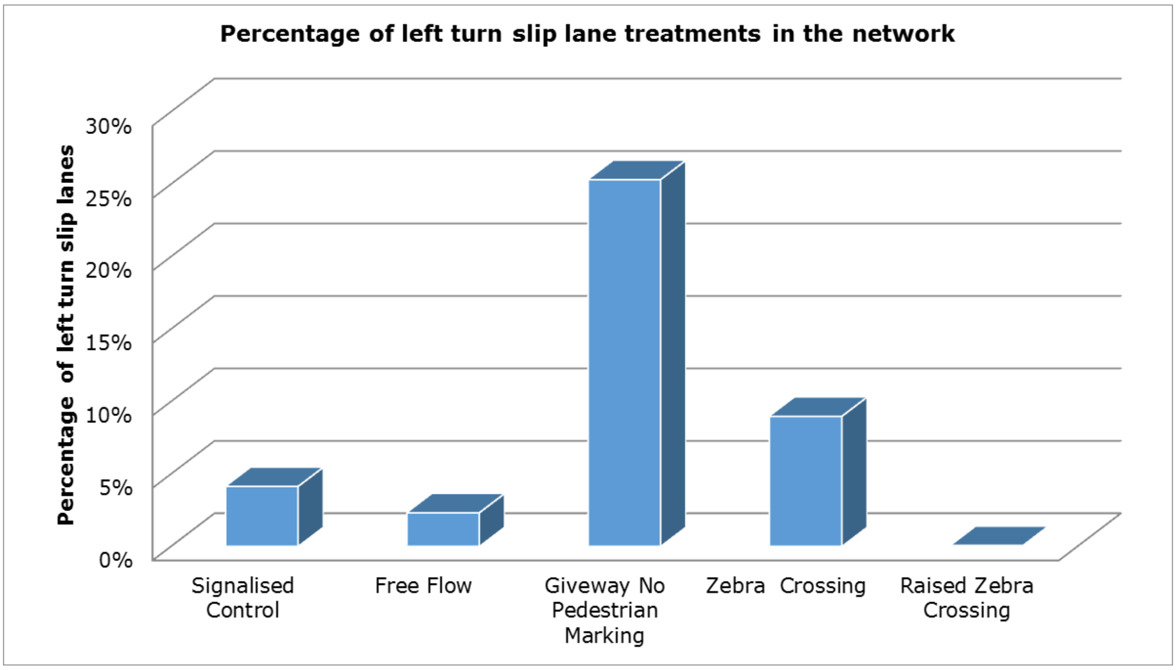


Figure 3-12 Distribution of the percentage of left turn slip lane treatments

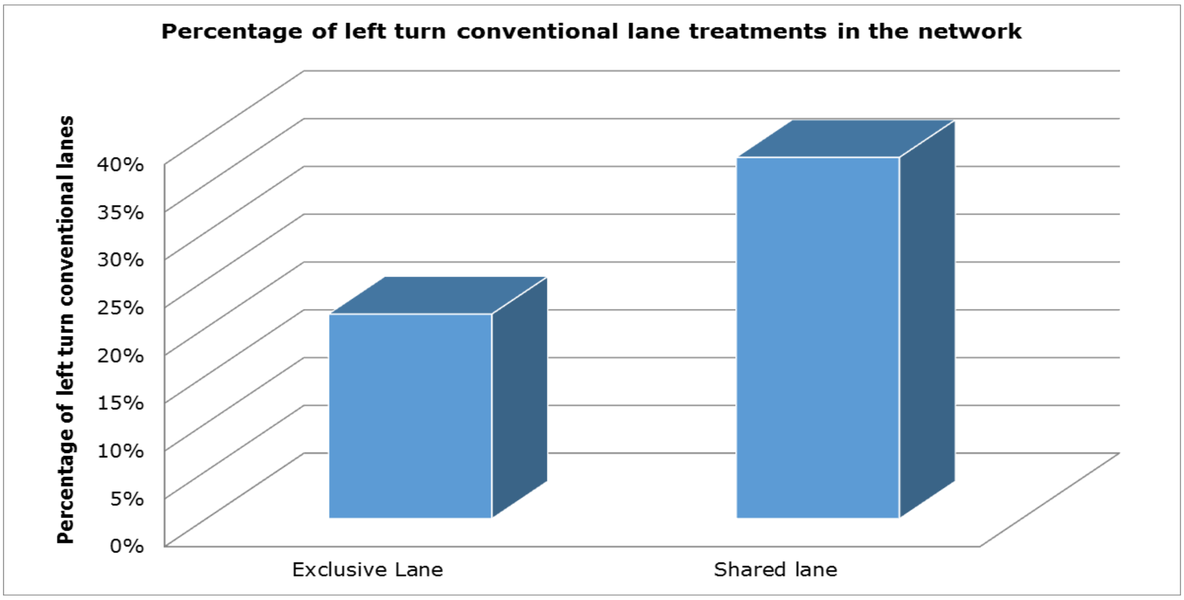


Figure 3-13 Distribution of the percentage of conventional left turn lanes

Figure 3-14 depicts the distribution of left turn slip lanes in comparison with the left turn conventional lanes. The greatest proportion of the left turn treatments was the left turn conventional lane 59%; the left turn slip lane was present at 41% of the approaches in the signals network.

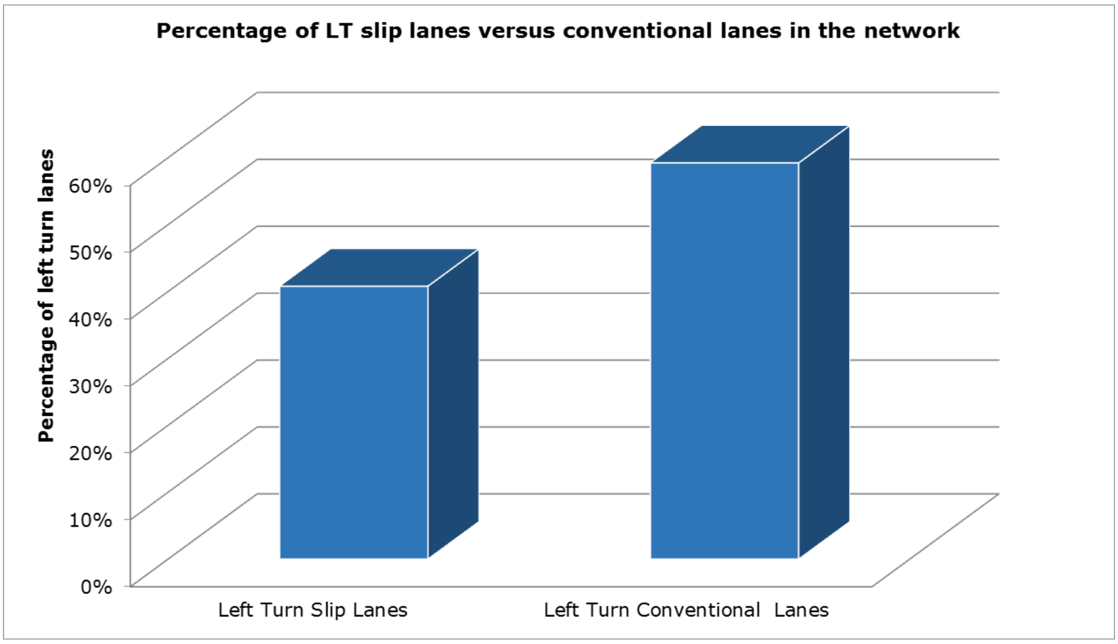


Figure 3-14 Distribution of the percentage of left turn slip lanes versus conventional lanes in the signals network

3.4 Crash data collection

3.4.1 General

The crash data was required to perform the safety analysis task of this study. Generally, there are two methods to query crashes from the NZ Transport Agency Crash Analysis System (CAS):

- The first method is to select a road or intersection(s) individually within a territorial local authority (TLA) and request the crashes of that site. This method is considered impractical due to the large sample size required (number of signalised intersections) involved); or
- The second method is to select the whole TLA and acquire the whole crash data set for signals network. This was considered to be the most suitable way of collecting crash data for this study.

CAS was interrogated for all reported crashes, for signalised intersections located in the Auckland region, over the period 2010–2014 (inclusive). The coded crash report was obtained and the following variables included: severity (fatal, serious, minor, non-injury), year, vehicle movement code, speed limit, wet/dry, dark/light, junction type, traffic signal control type only, crash factor codes and road user types.

Two problems were faced during this stage of crash data collection:

- Firstly, to collect crashes at signalising intersection including those which occurred at left turn slip lanes; and
- Secondly, to identify left turn crashes from other crashes that were not related to left turn movements.

These two problems are explained in more detail in the following sections.

3.4.2 Crash data collection problem

During the initial investigation and checking of the obtained CAS crash dataset, it was found that most of the left turn slip lanes crashes were not included. This was due to many left turn slip lane crashes being coded as give-way control not as signal control, even where it is located within signalised intersections.

This was one of the limitations of the CAS. Therefore, the attributes of the second method have been modified to include the give-way control, to capture the left turn slip lane crashes. The crash data was re-interrogated again from CAS including give-way control, as well as signal control intersections. The English language and coded crash reports were obtained from CAS.

This was to ensure that all left turn slip lane crashes, whether they were controlled by give-way or signals, were captured in the crash reports for all signalised intersections. This then required irrelevant crash reports that occurred at intersections that were controlled by give-way (unsignalised intersections) to be eliminated from the dataset.

There were a total of 22,000 crash records obtained from CAS after inclusion of give-way controlled sites, for the five year period. One of the major problems faced was how to exclude all the crashes that did not occur at signalised intersections.

To solve this problem, two techniques were attempted:

- Using Excel spreadsheet macros, which failed as explained in the following section; and
- Using spatial analysis software (QGIS), which is the method used in this research.

3.4.2.1 Using Excel spreadsheet macros

Firstly, the Excel macro was used to comparing GPS coordinates (X and Y) of signalised intersections with the crash coordinates located within a certain range of the signalised sites and flag them as “true”, to keep these crash records. Conversely, flag the other crashes as “false” when the coordinates fell outside the range, to delete them manually in an Excel spread sheet as shown in the snapshot in Figure 3-15.

After manually checking, it was found that crash data were flagged incorrectly in many instances. Consequently, the technique was considered inappropriate to be carried on with.

GRP SITE EASTING	GRP SITE NORTHING	CRASH ROAD	CRASH DIST	CRASH DIRN	INTSN	SIDE ROAD	CRASH ID	CRASH DATE	CRASH DOW	CRASH TIME	MVMT	VEHICLES	CAUSES	OBJECTS STRUCK	ROAD CURVE	ROAD WET	LIGHT	WTHRa	JUNC TYPE	TRAF CTRL	ROAD MARK	SPD LIM	CRASH FATAL CNT	CRASH SEV CNT	CRASH MIN CNT	PERS AGE1	PERS AGE2	Within 50R of signalised intersection GPS
1748069	5912232	TITIRANGI ROAD			I	WEST LYNN ROAD	201032729	20/04/2010	Tue	2030	KB	CN1C	301B		E	D	TO	F	T	S	P	50	0	0	0			FALSE
1749431	5913916	TITIRANGI ROAD	10	S		GREAT NORTH RO	201034031	3/04/2010	Sat	1400	MB	CN1C	372B		R	D	B	F	X	T	R	50	0	0	0			TRUE
1749431	5913916	TITIRANGI ROAD			I	GREAT NORTH RO	201431016	14/02/2014	Fri	1824	FB	CW1C	181A 350A		E	W	O	F	T	G	C	50	0	0	0			TRUE
1749431	5913916	TITIRANGI ROAD			I	GREAT NORTH RO	201032582	10/04/2010	Sat	1845	FE	CN1C	181A		R	D	DO	F	X	T	C	50	0	0	0			TRUE
1749431	5913916	TITIRANGI ROAD			I	GREAT NORTH RO	201103408	25/06/2011	Sat	1433	FE	4N1C	331A		E	D	BF	F	X	T	C	50	0	0	1			TRUE
1749431	5913916	TITIRANGI ROAD			I	GREAT NORTH RO	201102471	3/05/2011	Tue	720	NA	4N1E	711B 718B		R	D	BN	F	X	T	C	50	0	0	1	61		TRUE
1749431	5913916	TITIRANGI ROAD			I	GREAT NORTH RO	201232047	6/03/2012	Tue	1516	LB	CN1CC	325B		E	D	BF	F	X	T	R	50	0	0	0			TRUE
1749431	5913916	TITIRANGI ROAD			I	GREAT NORTH RO	201130419	14/01/2011	Fri	1115	FB	CN1C	181A		R	D	BF	F	X	G	C	50	0	0	0			TRUE
1749431	5913916	TITIRANGI ROAD			I	RATA ST	201131744	1/03/2011	Tue	850	FD	CN1C	181A		R	D	BF	F	X	T	C	50	0	0	0			TRUE
1749245	5913590	TITIRANGI ROAD			I	MARGAN AVENUE	201035565	16/06/2010	Wed	1505	FE	CS1C	181A		E	D	BF	F	T	T	L	50	0	0	0			TRUE
1749245	5913590	TITIRANGI ROAD			I	MARGAN AVENUE	201241116	11/10/2012	Thu	1445	FE	CN1C	181A		R	D	BF	F	T	T	C	50	0	0	0			TRUE
1749245	5913590	TITIRANGI ROAD			I	MARGAN AVENUE	201303630	12/06/2013	Wed	2250	CB	CS1	197A	P	R	D	DO	F	T	T	C	50	0	0	2			TRUE
1749245	5913590	TITIRANGI ROAD			I	MARGAN AVENUE	201411627	22/03/2014	Sat	2350	JA	4S1C	322A 101B		E	D	DO	F	T	T	R	50	0	0	1			TRUE
1749245	5913590	TITIRANGI ROAD			I	MARGAN AVENUE	201140229	25/08/2011	Thu	1537	FE	CN1C	181A		R	D	ON	F	T	T	C	50	0	0	0			TRUE
1747965	5912509	TITIRANGI ROAD			I	PLEASANT ROAD	201041358	23/10/2010	Sat	300	DB	CS1	101A 111A 131A	T	E	D	DO	F	T	T	C	50	0	0	0			TRUE
1747965	5912509	TITIRANGI ROAD			I	PLEASANT ROAD	201105011	22/09/2011	Thu	1633	LB	PN1C	334A 357A 414A		R	D	OF	F	T	T	C	50	0	0	1			TRUE
1747965	5912509	TITIRANGI ROAD			I	PLEASANT ROAD	201133631	6/05/2011	Fri	1645	FE	CN1C	101A 331A		R	D	OF	F	T	T	C	50	0	0	0			TRUE
1747965	5912509	TITIRANGI ROAD			I	PLEASANT ROAD	201231613	18/03/2012	Sun	210	DA	MS1	102A 111A 131A		S	D	DO	F	T	T	R	50	0	0	0			TRUE
1747965	5912509	TITIRANGI ROAD			I	PLEASANT ROAD	201417528	13/11/2014	Thu	1728	LB	CN1C	303B		E	D	BF	F	T	T	C	50	0	0	1			TRUE
1747965	5912509	TITIRANGI ROAD			I	PLEASANT ROAD	201237944	11/09/2012	Tue	800	KB	CS1C	322A 514A 517A		E	W	O	F	T	T	C	50	0	0	0			TRUE
1747965	5912509	TITIRANGI ROAD			I	PLEASANT ROAD	201330920	17/01/2013	Thu	1700	LB	CN1C	303B 375B		R	D	OF	F	T	T	L	50	0	0	0			TRUE
1747965	5912509	TITIRANGI ROAD			I	PLEASANT ROAD	201337502	29/08/2013	Thu	2355	JA	CN1C	322A		E	D	DO	F	T	T	R	50	0	0	0			TRUE
1747965	5912509	TITIRANGI ROAD			I	PLEASANT ROAD	201338968	6/10/2013	Sun	1430	FE	4S1C	181A		R		F		T	T	C	50	0	0	0			TRUE
1747965	5912509	TITIRANGI ROAD			I	PLEASANT ROAD	201356036	12/10/2013	Sat	1642	DB	CE2	133A 420A	C	R		F		T	G	C	50	0	0	0			TRUE

Figure 3-15 A snapshot of the Excel macro method

3.4.2.2 Using spatial analysis software (QGIS)

The second technique was attempted utilising QGIS software. Four steps were conducted in this method, as follows:

- The first step was to create two map layers: the signalised intersections layer and the crashes data layer;
- The second step was to create another buffer zone layer of 50m around each signalised intersection;
- Then the third step was to run an intersecting query between the crash layer and the buffer zone layer; and
- Finally, the crashes within that 50m buffer range for each signalised intersection was captured and the others were rejected as indicated in a snapshot in Figure 3-16.

Figure 3-16 A snapshot of the QGIS method

A manual check was conducted on a few intersections to ensure there were no data missing, or incorrectly removed from the list or added to the list. It was easy to check crash data and signalised intersections visually on a map compared to the Excel macro method. This method successfully eliminated all crash data that occurred at give-way controlled intersections.

After the data clearing, the crash data records were reduced to approximately 8,000 crash records (over the five year period).

The last step of this technique was to run another query via QGIS to assign the crash data records that occurred at signalised intersections to their associated intersection name and number. Then these data were extracted into a spreadsheet to form a combined database of intersection data and crash data.

This task was necessary for the next step of the crash analysis. It is worth noting that the QGIS method was superior to the Excel macro method in doing these tasks.

Figure 3-17 shows a snapshot of the combined database of crash data (highlighted in green) assigned to the intersection data (highlighted in red) with intersection name and number extracted from QGIS.

AREA	Intersection	Description	CRASH ROAD	CRASH DIST	CRASH DIRN	INTSN	SIDE ROAD	CRASH ID	CRASH DATE	CRASH DOW	CRASH TIME	MVMT	VEHICLES	CAUSES	OBJECTS ST	ROAD CURVE	ROAD WET	LIGHT	WTHRa	JUNC TYPE	TRAF CTRL	ROAD MARK	SPD LIM	CRASH FATA	CRASH SEV	CRASH MIN	PERS AGE1
North	1101	WAIRAU RD - ARCHERS RD	WAIRAU ROAD			I	ARCHERS ROAD S	201239186	26/09/2012	Wed	710	AA	VS1C	372A		R	D	BF	F	X	T	C	50	0	0	0	
North	1102	WAIRAU RD - TRISTRAM AVE - HILLSIDE RD	TRISTRAM AVENUE	10	E		WAIRAU ROAD	201236834	5/08/2012	Sun	1338	AA	CW14	381A		R	D	BF	F	X	T	R	50	0	0	0	
North	1102	WAIRAU RD - TRISTRAM AVE - HILLSIDE RD	TRISTRAM AVENUE			I	WAIRAU ROAD	201410996	12/03/2014	Wed	2100	AA	MW1C	197A 372B		R	D	BF	F	T	T	C	50	0	0	1	
North	1102	WAIRAU RD - TRISTRAM AVE - HILLSIDE RD	WAIRAU ROAD	10	S		TRISTRAM AVENUE	201332334	6/04/2013	Sat	1400	AA	CN1C	372A 372B		R	D	OF	F	X	T	C	50	0	0	0	
North	1201	ANZAC ST - AUBURN ST	ANZAC ST	10	E		AUBURN ST	201332342	6/03/2013	Wed	1800	AA	CN1C	143B 372B		R	D	BF	F	X	T	P	50	0	0	0	
North	1201	ANZAC ST - AUBURN ST	ANZAC ST			I	AUBURN ST	201449887	2/12/2014	Tue	1050	AA	CE1C	372A 671A	M	R	D	BF	F	X	T	C	50	0	0	0	
North	1205	TAHAROTO RD - SHAKESPEARE RD - WAIRAU RD	TAHAROTO ROAD	5	E		SHAKESPEARE ROAD	201044506	12/12/2010	Sun	1300	AA	CW1C	372A		R	D	BF	F	M	T	C	50	0	0	0	
North	1208	IORTHCOTE RD - SUNNYBRAE RD - AKORANGA D	AKORANGA DRIVE	10	S		NORTHCOTE ROAD	201235094	22/05/2012	Tue	1630	AA	CN1C	372A		R	D	BF	F	X	T	C	50	0	0	0	
North	1208	IORTHCOTE RD - SUNNYBRAE RD - AKORANGA D	SUNNYBRAE ROAD	10	N		NORTHCOTE ROAD	201030248	14/01/2010	Thu	1019	AA	CN1V	184A 357A		R	D	B	F	X	T	R	50	0	0	0	
North	1301	LAKE RD - BAYSWATER AVE - WILLIAMSON AVE	LAKE ROAD	10	N		BAYSWATER AVENUE	201142618	18/11/2011	Fri	1545	AA	CN14	386A		R	D	BF	F	X	T	N	50	0	0	0	
North	1301	LAKE RD - BAYSWATER AVE - WILLIAMSON AVE	LAKE ROAD			I	BAYSWATER AVENUE	201031336	18/02/2010	Thu	720	AA	CN1C	372A		R	D	BF	F	X	T	C	50	0	0	0	
North	1402	GLENFIELD RD - MANUKU RD - HOGANS RD	GLENFIELD ROAD			I	MANUKA ROAD	201233706	26/03/2012	Mon	840	AA	TS1C	372A		R	D	BF	F	X	T	C	50	0	0	0	
North	1506	NEWA RD - BIRKENHEAD AVE - HIGHBURY BYPA	BIRKENHEAD AVENUE			I	ONEWA ROAD	201333914	31/05/2013	Fri	1143	AA	CS1T	372A 671A		R	D	OF	F	X	T	C	50	0	0	0	
North	1506	NEWA RD - BIRKENHEAD AVE - HIGHBURY BYPA	HIGHBURY BYPASS			I	BIRKENHEAD AVENUE	201357932	15/10/2013	Tue	800	AA	CE1C	372A		R	D	OF	F	X	T	C	50	0	0	0	
North	1604	EAST COAST RD - SUNRISE RD	EAST COAST ROAD			I	SUNRISE AVENUE S	201040457	14/10/2010	Thu	1234	AA	TW1C	129A 386A		R	D	B	F	X	T	C	50	0	0	0	
North	1609	AST COAST RD - PONDEROSA DR - MULGAN WA	EAST COAST ROAD	10	S		MULGAN WAY	201448456	20/11/2014	Thu	1000	AA	VS1T	381A		R	D	BF	F	T	G	C	50	0	0	0	
North	1610	AST COAST RD - OTEHA VALLEY RD - CARLISLE	EAST COAST ROAD			I	OTEHA VALLEY ROAD	201236263	19/07/2012	Thu	1620	AA	CS1C	386A 402A 434A		R	D	BF	F	X	T	C	50	0	0	0	
North	1610	AST COAST RD - OTEHA VALLEY RD - CARLISLE	OTEHA VALLEY ROAD	5	W		EAST COAST ROAD	201131182	5/03/2011	Sat	1018	AA	4E1C	372A		R	W	ON	L	T	T	C	50	0	0	0	
North	1614	CONSTELLATION DR - APOLLO DR	APOLLO DRIVE	5	N		CONSTELLATION DRIV	201240218	3/11/2012	Sat	1750	AA	CS1C	372A 671A		R	D	OF	F	X	T	C	50	0	0	0	
North	1643	SH17 - BUSH RD - MERCARI WAY	ALBANY EXPRESSWAY			I	MERCARI WAY	201235844	16/07/2012	Mon	1215	AA	CW1C	381A		R	W	OF	L	X	T	C	80	0	0	0	
North	1647	SH17 - OTEHA VALLEY RD EXTN (Albany Village	ALBANY EXPRESSWAY			I	DAIRY FLAT HIGHWAY	201335092	20/06/2013	Thu	1600	AA	CS1T	372A		R	W	OF	L	X	T	C	50	0	0	0	
North	1647	SH17 - OTEHA VALLEY RD EXTN (Albany Village	DAIRY FLAT HIGHWAY			I	OTEHA VALLEY ROAD	201141352	4/11/2011	Fri	2141	AA	CS1C	372A		R	D	DO	F	X	T	P	60	0	0	0	
North	1810	1 (HIBISCUS COAST HWAY) / WHANGAPARAOA	BISCUS COAST HIGHW	10	S		WHANGAPARAOA ROA	201140549	19/09/2011	Mon	1745	AA	CN1C	372A		R	D	O	F	X	T	P	80	0	0	0	
North	1902	SH1 - SH18 - CONSTELLATION DRIVE	18/0/0.109			I	UPPER HWY OFF SBD	201444881	17/11/2014	Mon	1540	AA	CW1T	372A		R	D	OF	FS	X	T	R	80	0	0	0	
North	1902	SH1 - SH18 - CONSTELLATION DRIVE	18/0/0.109			I	JPPER HWY ON SBD V	201437150	21/05/2014	Wed	1230	AA	VE1C	381A		R	D	BF	F	T	T	C	50	0	0	0	
North	1904	SH18 - ALBANY HIGHWAY INTERCHANGE	ALBANY HIGHWAY			I	BANY HIGHWAY ON E	201033824	14/04/2010	Wed	1545	AA	CS1C	372A		R	D	BF	F	X	T	R	50	0	0	0	
North	1913	ESMONDE RD - FRED THOMAS DR	ESMONDE ROAD	10	E		FRED THOMAS DRIVE	201241928	30/12/2012	Sun	1504	AA	CW1C	209A 372A		R	D	O	F	T	T	R	60	0	0	0	
North	1913	ESMONDE RD - FRED THOMAS DR	ESMONDE ROAD	10	W		FRED THOMAS DRIVE	201231648	10/03/2012	Sat	1230	AA	ME1C	184B		R	D	O	F	T	T	N	80	0	0	0	
North	1914	MONDE RD - AKORANGA BUS STATION ENTRAN	ESMONDE ROAD			I	ACCESS ROAD	201132853	5/03/2011	Sat	600	AA	BE1C	198A		R	W	OO	F	T	T	C	80	0	0	0	
North	1936	SH1 / WHITAKER RD (WARKWORTH)	1N/363/2.989			I	WHITAKER ROAD	201332817	17/03/2013	Sun	1300	AA	CN14	372A		R	W	OF	L	T	T	C	60	0	0	0	

Figure 3-17 Snapshot of the crash data assigned to the intersection data

3.4.3 Left turn crashes identification problem

Another major difficulty of the study was to identify left turn crashes at signalised intersections, particularly crashes that happened at slip lanes and those that occurred at conventional lanes.

This was a key problem as CAS does not have any mechanism in place to classify or identify the left turn crashes at signalised intersections. In addition, there is no such category or subcategory to ascertain crashes occurring at left turn slip lanes. In other words, there is no such crash code(s) that can be selected from the CAS vehicle movement coding sheet to specifically identify the left turn crashes. The CAS vehicle movement codes list is presented in Appendix A.

Also, it was difficult to use the standard crash data reports direct from CAS to identify left turn slip lane crashes. This was due to the crash movement codes at these locations usually reflecting what actually occurred in the crash instead of what the road layout was. Accordingly, this makes it problematic to reliably identify them using the crash data alone.

Hence, the only way to identify these left turn slip lane crashes was to interrogate and examine each crash record individually by sighting the traffic crash report TCR. This approach was considered impractical, and unachievable to do so, due to the large number of crash records that required sighting the TCRs (approximately 8,000 crash records each with approximately four pages for = 32,000 pages).

It was decided to communicate with the CAS specialists at NZTA. A number of meetings and discussions with the CAS Manager and CAS Coder at Auckland and Christchurch NZTA offices were conducted. This was to find out a practical mechanism to identify possible left turn crashes at signalised intersections.

The aim was to reduce the number of crash records (8,000 crash records) that required investigation, by eliminating the crash movement codes that were unlikely to occur at signalised intersections or involving left turn movements. This will reduce the total number of crash records that needed to be investigated and analysed for the purpose of this research.

3.4.4 Left turn crashes identification methodology

There were two main attempts carried out to identify left turn crashes from the CAS vehicle movement codes, as follows:

- Split CAS vehicle movement codes into two groups; and
- Eliminate CAS vehicle movement codes irrelevant to left turn movements.

The detail of each attempt is explained in the following sections.

3.4.4.1 Split CAS vehicle movement codes into two groups

The first attempt considered was to exclude crashes that most likely do not involve left turning movements. Next was to split the remaining crash movement codes into two groups: one group for slip lanes and the other for conventional lanes. The initial thought of this approach was to find a practical and easy way of splitting the slip lane crashes from conventional lane crashes, without a need to sight each TCR.

After further discussions with the CAS specialist, it was decided that this approach was not accurate enough as many of crash movement codes excluded could be related to left turn movements. Moreover, some crash movement codes could have occurred at both slip lane as well as conventional lane; therefore, it was inappropriate to separate them into two groups.

A version of the modified vehicle movement codes was developed and presented, as shown in Appendix B.1.

3.4.4.2 Eliminate CAS vehicle movement codes irrelevant to left turn movements

In the second attempt, it was acknowledged that in order to identify left turn crashes, TCR records had to be sighted, as well as ignoring splitting crash vehicle movement codes into two groups: conventional lane crash codes and slip lane crash codes.

Initially CAS vehicle movement codes were examined, and codes that were unlikely to involve left turn movement crashes and/or do not usually occur at signalised intersections were excluded, as shown in Appendix B.2. The modified CAS vehicle movement codes were established and presented in Appendix B.3.

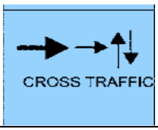
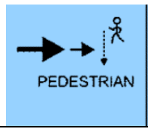
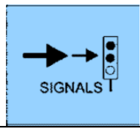
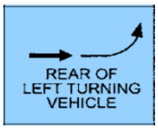
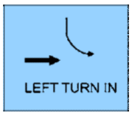
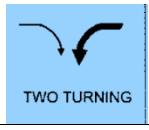
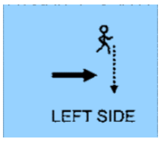
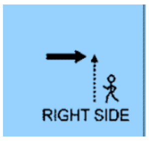
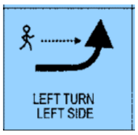
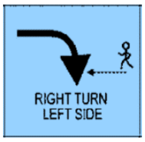
Furthermore, a pilot study was conducted on 10% of the crash records obtained from CAS, with the corresponding excluded CAS vehicle movement codes, as well as sighting the relevant TCR reports that were required to ensure that the crash movement codes that were omitted do not involve left turn movements. This was necessary to ensure the methodology was sound and appropriate.

As a result, the CAS vehicle movement codes list was modified and the following crash movement codes were excluded: DA, DC, FA, FD, FF, GB, KB, ND, NF, NG, and NO. The final version of the CAS vehicle movement codes was established and presented in Appendix B.4.

3.4.5 Left turn crash movement codes

The final list of CAS vehicle crash movement codes that were most likely to involve left turning crashes are shown in Table 3-3. This list will be used in the crash analysis tasks. It was obtained from the final CAS vehicle crash movement codes presented in Appendix B.4.

Table 3-3 List of most likely left turn crash movement codes

Crash Group Type	Crash Code			
Cornering	DB			
				
Rear End	FB	FC	FE	
				
Turning Versus Same Direction	GA	GF		
				
Merging	KA	KC		
				
Pedestrian Crossing Road	NA	NB	NC	NE
				

The list of the 8,000 crash records was filtered using the final CAS vehicle movement codes as depicted in Table 3-3. As a result, the total number of crash records was reduced to approximately 3,000 records, for the five year period.

This data formed the **main crash dataset** which used for the overall crash analysis, detailed analysis and subsequent additional pedestrian crash analysis.

3.5 Summary

The study methodology and data collection used for this research involved two key steps:

- **Signalised intersection data:** This included splitting the whole signals network of 625 signalised sites into approaches; then these approaches were categorised into the various left turn treatment types; and

- **Crash data for each signalised site:** This involved obtaining crash data for the whole signals network, identifying crashes involving left turn movements and assigning the crashes to each of the approaches.

The key summary of methodology and data collection is presented in the following sections.

3.5.1 Signalised intersection data

The 625 signalised intersections were divided into 1818 approaches that allow left turns. These approaches were categorised into the seven left turn treatments as follows:

- Exclusive conventional lane (389, 21%);
- Shared conventional lane: shared through and left, or shared left and right (688, 38%);
- Slip lane signal control, with signalised pedestrian crossing marking (74, 4%);
- Slip lane with give-way control, with zebra pedestrian crossing marking (160, 9%);
- Slip lane with give-way control, with no pedestrian crossing marking (463, 25%);
- Slip lane with give-way control, with zebra pedestrian crossing marking on raised table (1,0%); and
- Slip lane free flow, with no control (43, 2%).

It is recommended that the Road Controlling Authorities develop and maintain a record of left turn lanes data, particularly the left turn slip lanes and the existing facilities.

3.5.2 Crash data for each signalised intersection

CAS was interrogated for all reported crashes for the 625 signalised intersections, over the period 2010–2014 (inclusive). The coded crash report was obtained for these crashes. Two problems were encountered during this stage of crash data collection due to CAS limitations:

- The first problem was to collect crashes at signalising intersection including those which occurred at left turn slip lanes; and
- The second was to identify left turn crashes from other crashes that were not related to left turn movements.

To overcome the first problem, the QGIS technique was used to collect crashes that were related to signalised intersections and allocated these crashes to their corresponding intersection name and number. The second problem was solved by modifying the crash movement codes several times to eliminate the crashes that were unlikely to occur at signalised intersections or involving left turn movements. Lastly, the final list of modified crash movement codes was developed.

3.5.3 Improvement to CAS

During the process of the crash data collection stage, several recommendations to improve CAS were identified that would make the data collection stage for future researchers easier. These improvements include:

- The method to query signalised intersection crashes in CAS should include give-way slip lanes as well as signalised slip lanes;
- Developing a mechanism in CAS to identify left turn crashes from other crashes. For example, develop a code for left turn crashes and sub-codes for slip lane left turn crashes, and conventional left turn crashes. Additionally, the left turn slip lane crashes can be classified into: give-way signalised single lane or double lanes, zebra crossing, zebra on raised table and free flow. Similarly, the left turn conventional lane crashes can be classified into exclusive single or double left turn, shared left and through or shared left and right turn; and
- Developing a method/system to identify crash location by approach and by lane configuration in an easy way without the need to review the TCR. For example, whether the crash occurred in a through, right or left turn lane.

4 Overall Crash Analysis Results

The purpose of the overall crash analysis task was to get a general understanding of the safety performance of the different left turn treatment types for the entire signals network.

4.1 General

It was considered infeasible to conduct the overall crash analysis task for the full 5 year period. Due to the large number of crash records and the extensive manual interrogation required. Consequently, one year of crash analysis was used for the overall crash analysis task: that was for the year 2014.

The obtained crash list has been filtered using the CAS vehicle crash movement codes, as listed in Table 3-3. A total of 567 crash records remained after filtering. Then a crash query was created in CAS using a macro file, to download the TCRs for all 567 crash records. The TCR reports were obtained from CAS and encompassed approximately 1700 pages for one year only.

The next step was to conduct an extensive manual examination of every crash record to identify the left turning crashes and to exclude those that were not related to the left turn movements.

The data resulting from the initial investigation is summarised in Table 4-1.

Table 4-1 Initial results of crash data analysis

Type of crashes	No of crashes
Crashes involving left turn movements	230
Subtotal crashes included	**230**
Crashes not related to left turn movements	331
TCR was not available	6
Subtotal crashes excluded	**337**
Total crashes	**567**

Table 4-1 indicates that there were 230 crashes to be further analysed which involved left turn movements. A total of 337 crashes were excluded from the analysis, as these crashes were not related to left turn movements or their TCRs were not available.

Two methods were used to analyse left turn crashes:

- Un-classified left turn crash analysis; and
- Classified left turn crash analysis.

The details of each method is described in the following sections.

4.2 Un-classified left turn crash analysis

In this method, the crashes were analysed without classifying them into the various left turn treatment types. These crashes were analysed by: severity, movement types, contributing factors, light conditions, road surface conditions, month, day and time, sex and age group, vehicle type and object struck.

4.2.1 Left turn crash data by severity

Table 4-2 indicates that majority of left turn crashes (86%) were non-injury crashes and 13% were minor injury crashes.

Table 4-2 Left turn crashes by severity

Crash Severity	Number	%
Fatal	1	0.4
Serious	1	0.4
Minor Injury	31	13.5
Non-injury	197	85.7
TOTAL	230	100

4.2.2 Left turn crash data by movement types

Table 4-3 shows that the vast majority of crashes were rear-end/obstruction (70%) and 16% were crossing/turning crashes. There were only 3% pedestrian crashes. Given this small percentage, it appears that crashes involving pedestrians were not a major issue.

Table 4-3 Left turn crashes by movement types

Crash Movement Type	Number	%
Overtaking Crashes	0	0
Straight Road Lost Control/Head On	0	0
Bend - Lost Control/Head On	26	11
Rear End/Obstruction	160	70
Crossing/Turning	37	16
Pedestrian Crashes	**7**	**3**
Miscellaneous Crashes	0	0
TOTAL	230	100

4.2.3 Crash data by factors

The contributing factors for left turn crashes are shown in Figure 4-1. The two key contributing factors are incorrect lane position (34%) and poor observation (24%). Failed to give-way/stop and poor judgement were found to have the same frequency (10%).

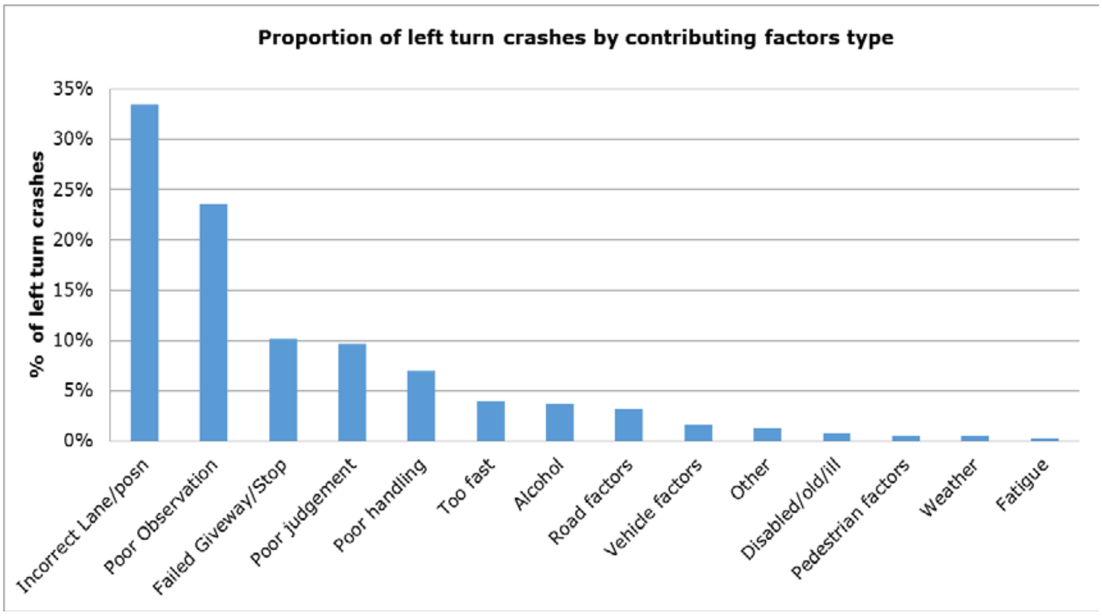


Figure 4-1 Distribution of left turn crashes by factors

4.2.4 Left turn crash data by light condition

Table 4-4 illustrates that 76% of left turn crashes happened in daylight condition. The remaining crashes occurred in dark or near dark conditions (24%).

Table 4-4 Left turn crashes by light conditions and severity

Natural Light Conditions	Injury	Non-injury	Total	%
Light/overcast	27	146	173	75.2
Dark/twilight	6	48	54	23.5
Unknown	0	3	3	1.3
TOTAL	33	197	230	100

Table 4-5 depicts the light condition for all signalised intersection crashes.

Table 4-5 All crashes of all signalised intersection by light conditions and severity

Natural Light Conditions	Injury	Non-injury	Total	%
Light/overcast	1041	4137	5178	64.7
Dark/twilight	696	2125	2821	35.3
TOTAL	1737	6262	7999	100.0

From both Table 4-4 and Table 4-5 the comparison analysis showed that the crashes that occurred in dark/twilight conditions were within 12% difference; in fact, they were less for left turn crashes (23% versus 35%). This suggests that the light conditions were not a crucial factor for the left turn crashes.

4.2.5 Left turn crash data by road surface condition

Table 4-6 displays the road condition patterns for 226 left turn crashes. Predominantly, crashes occurred in dry conditions (79%), with 21% of crashes occurring on wet roads during or after rain.

Table 4-6 Left turn crashes by road surface conditions and severity

Road Surface Conditions	Injury	Non-injury	Total	%
Dry	25	153	178	79
Wet	8	40	48	21
Ice/snow	0	0	0	0
TOTAL	33	193	226	100

Table 4-7 indicates the road conditions for all signalised intersections crashes.

Table 4-7 All crashes of all signalised intersections by road surface conditions and severity

Road Surface Conditions	Injury	Non-injury	Total	%
Dry	1320	4740	6042	76
Wet	434	1490	1924	24
Ice/snow	0	0	0	0
TOTAL	1736	6230	7966	100

The comparison of road condition pattern of the left turn crashes with all signalised intersection crashes show that the proportion of crashes was similar, with only 3% difference, which is inconsiderable. This suggests that the road conditions factor is not important for the left turn crashes.

4.2.6 Left turn crash data by month, day and time

Figure 4-2 depicts the monthly distribution of left turn injury and non-injury crashes. The greatest percentage of crashes occurred in September and the least in April. The monthly proportional did vary, ranging from 9% to 28%.

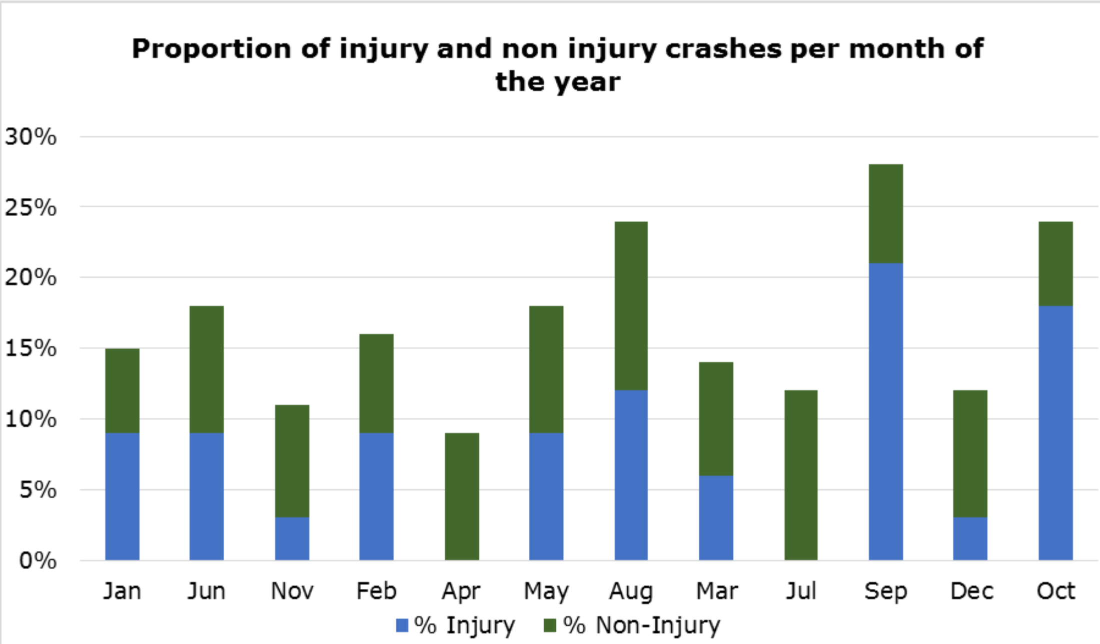


Figure 4-2 Distribution of left turn crashes per month of the year

Figure 4-3 shows the left turn crash frequencies by day of the week. Frequencies generally varied from Monday to Sunday, with Friday having the greatest proportion (27%) and Sunday the lowest (6%).

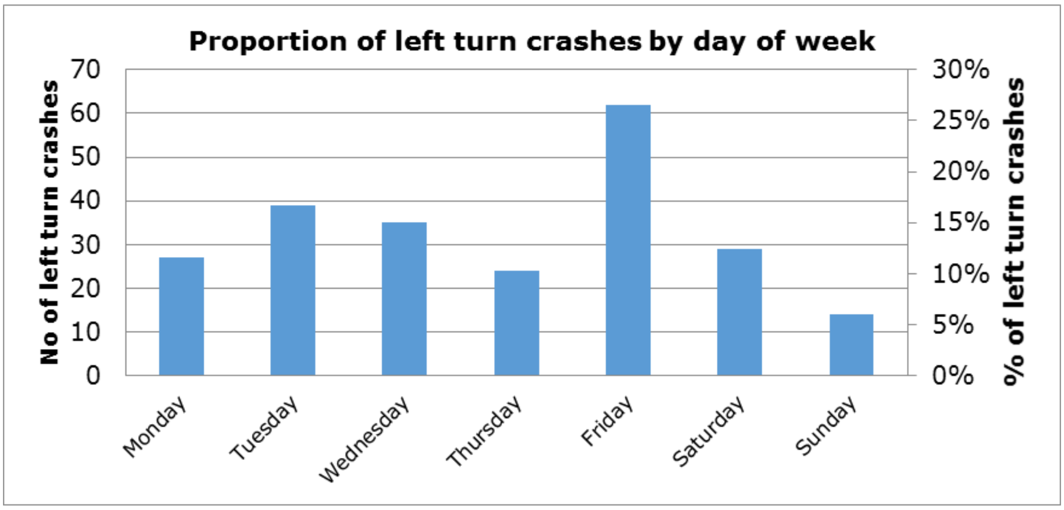


Figure 4-3 Distribution of left turn crashes per day of the week

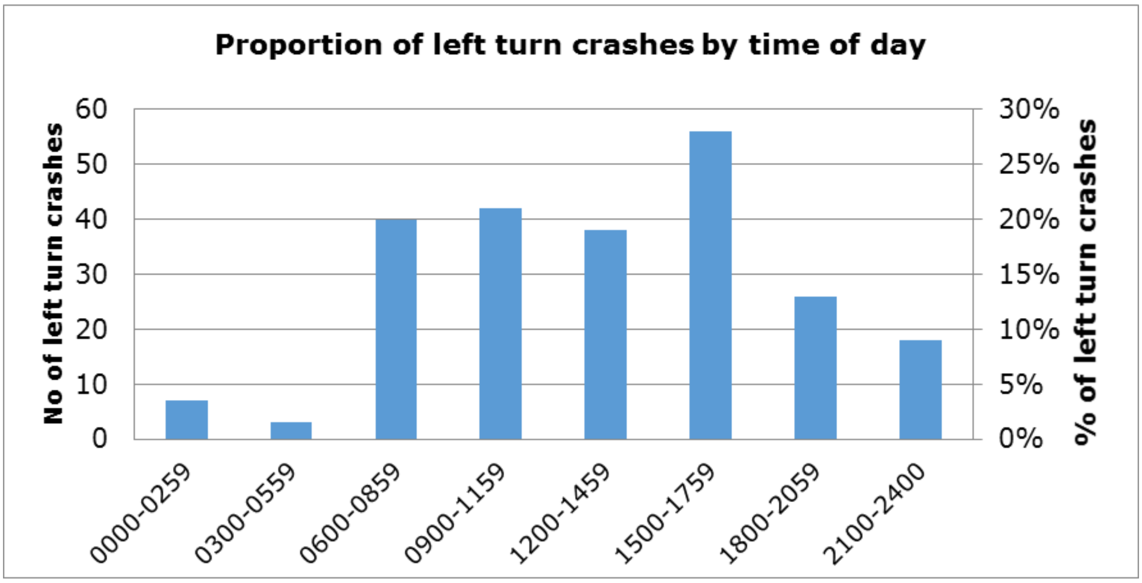


Figure 4-4 Distribution of left turn crashes per time of day

The distribution of the time of each of the 230 left turn crashes is presented in Figure 4-4, in three hourly time intervals. The frequency of crashes steadily increased from 6am to 17:59, peaking between 15:00 and 17:59. Almost 50% of crashes occurred from 6am to 3pm.

Figure 4-5 indicates that the majority of left turn crashes occurred during PM peak (31%) time (3.30pm to 7 pm), followed by AM peak (28%).

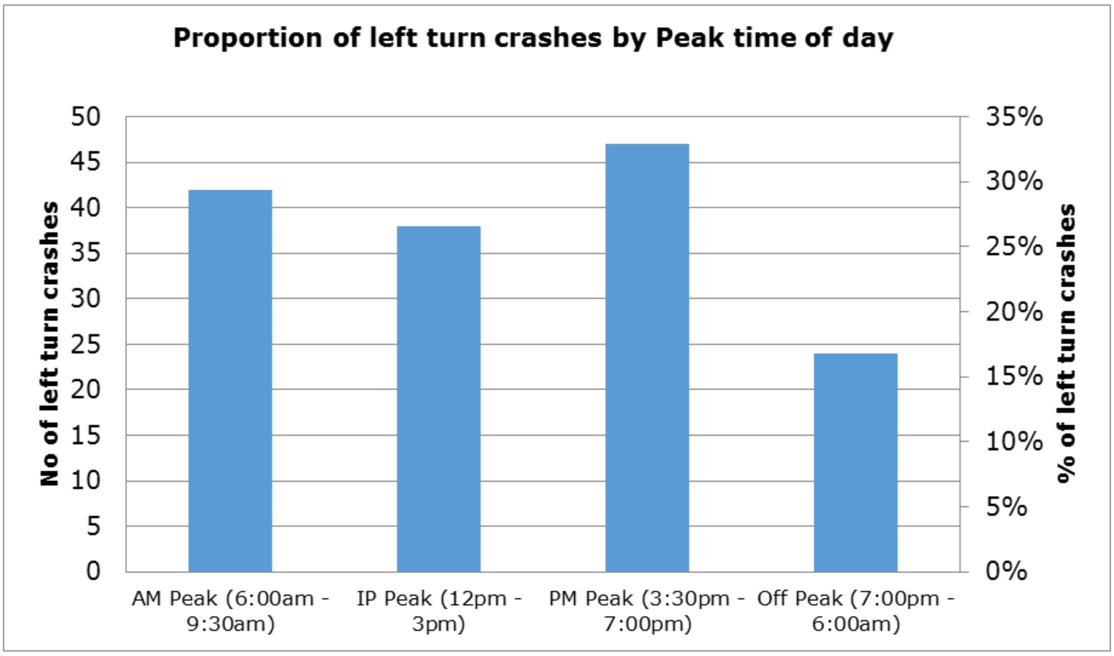


Figure 4-5 Distribution of left turn crashes per peak time of day

4.2.7 Crash data by sex and age group

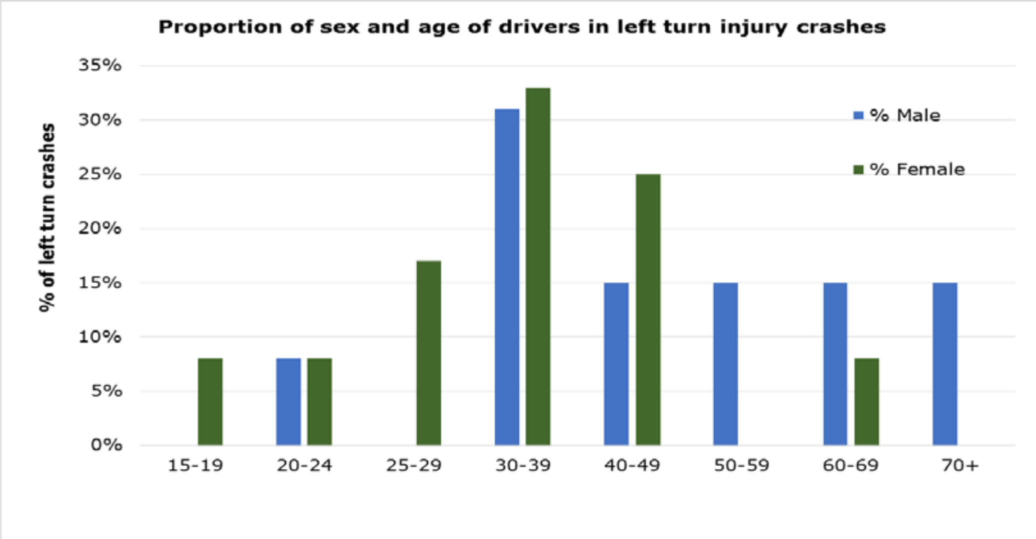


Figure 4-6 Distribution of left turn injury crashes by sex and age group

Figure 4-6 shows the sex and age group distribution of drivers involved in the left turn injury crashes only. The non-jury crashes were not available in CAS. Crash frequencies for male drivers aged 40 years and over, remained the same with increasing age (15%); on the contrary, female drivers generally decreased.

The graph indicates that none of the female and male drivers in the same age group have the same proportion of crashes. Moreover, the graph does not indicate any distinct patterns except that the male and female drivers aged 30-39 year group appear to have a higher crash frequency (31% and 33%) compared to other age groups. Moreover, the male and female drivers experienced the same share of number of left turn crashes.

4.2.8 Crash data by vehicle type

The types of vehicles involved in the left turn crashes are depicted in Figure 4-7. The greatest proportions of these crashes involved cars (83%) or car derivatives. Other vehicle types like motorcycle and bicycle presented a very low proportion of crashes, with only 1% each.

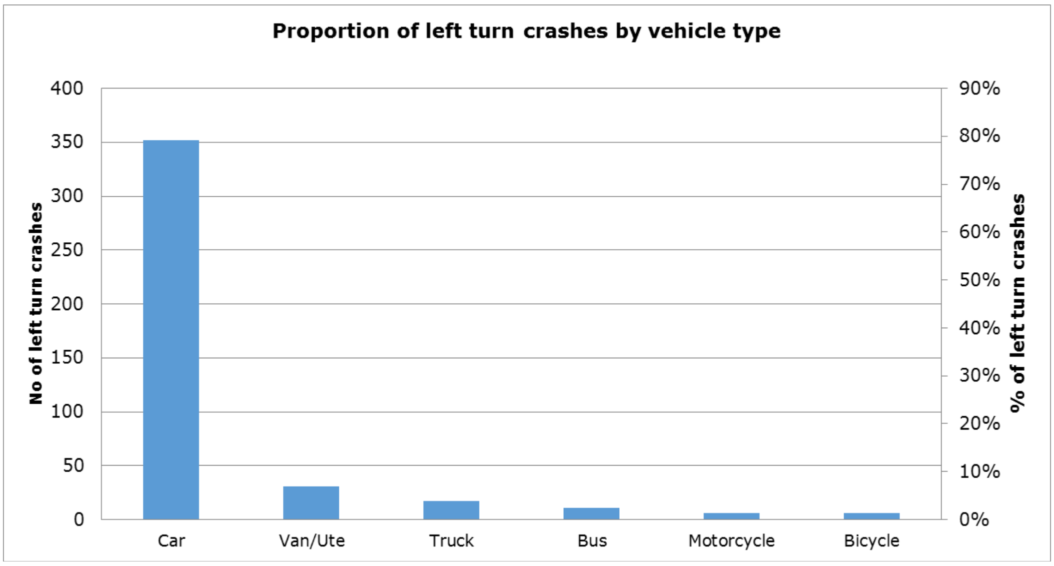


Figure 4-7 Distribution of left turn crashes by vehicle type

4.2.9 Crash data by object struck

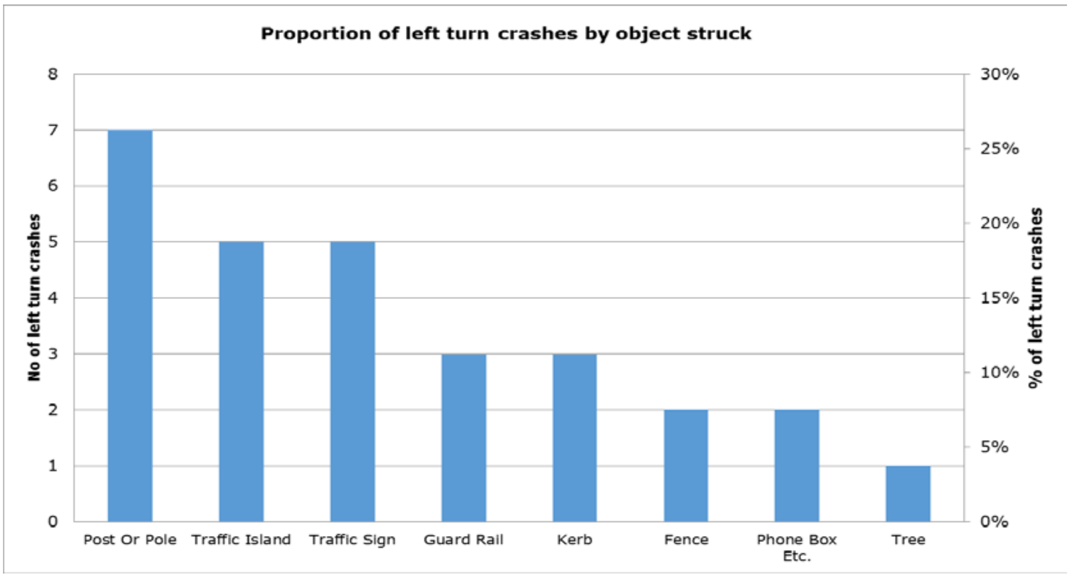


Figure 4-8 Distribution of left turn crashes by object struck

As shown in Figure 4-8, the highest three left turn crash frequency by object struck involved a post or pole (25%), traffic island (18%) and traffic sign (18%).

4.3 Classified left turn crash analysis

In this method, the crashes were analysed and classified into the various left turn treatment types for all road users together and for pedestrians only.

4.3.1 General

The 230 left turn crashes were analysed further by classifying crashes according to their left turn lane treatment types. A manual check for each crash record was performed including reviewing TCR along with information from the signalised intersection database (left turn classification spreadsheet). This was to identify whether the crash occurred at a slip lane or a conventional lane and the type of facility that existed. Then every crash record was coded with three letters as presented in Table 4-8.

Table 4-8 List of left turn type and code

Code	Left Turn Type
ECL	Exclusive Conventional Lane
SCL	Shared Conventional Lane
FSL	Free Slip Lane
GSL	Give-way Slip Lane
ZSL	Zebra Slip Lane
SSL	Signalised Slip Lane

The detailed crash report for the 230 crashes including the left turn crash treatments is presented in Appendix C.

Figure 4-9 shows the distribution of the left turn crashes by the left turn treatment types.

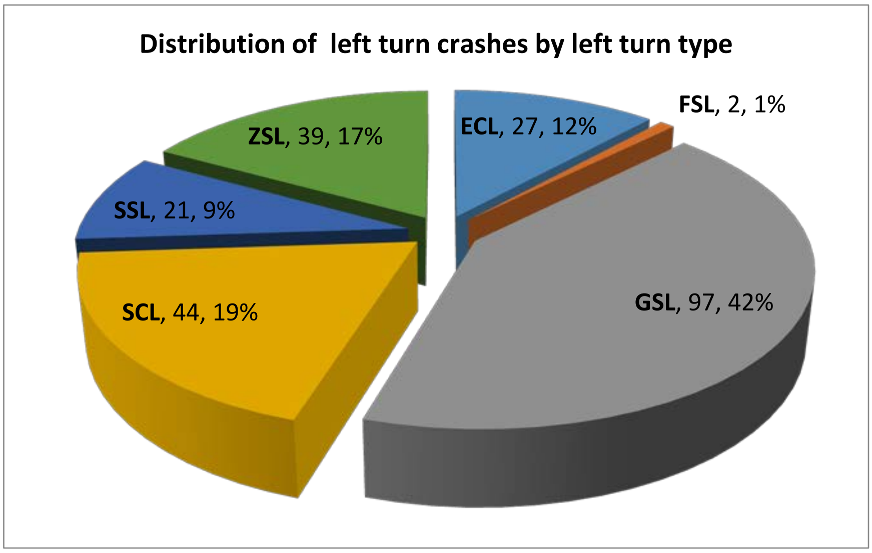


Figure 4-9 Distribution of left turn crashes by left turn type

4.3.2 Left turn crashes by treatments and frequency –all road users

Table 4-9 summarises the crash sites by categorising the several types of left turn slip lanes and the left turn conventional lanes. Left turn slip lanes experienced 69% of the crashes at 41% of the sites, while left turn conventional lanes experienced 31% of the crashes at 59% of the sites.

Table 4-9 Left turn crashes by type and treatment frequency in the signals network

<table>
<tr><th colspan="2" rowspan="2">Left Turn Treatment Type</th><th colspan="2">Left turn treatments frequency in the network</th><th colspan="2">Left turn crash types –all road users</th></tr>
<tr><th>No. of left turn treatments in the signal network</th><th>% of left turn treatments in the signal network</th><th>No. of left turn crashes</th><th>% of left turn crashes</th></tr>
<tr><td rowspan="6">Slip Lane</td><td>Signalised</td><td>75</td><td>4%</td><td>21</td><td>9%</td></tr>
<tr><td>Free Flow</td><td>42</td><td>2%</td><td>2</td><td>1%</td></tr>
<tr><td>Give-way No Pedestrian Marking</td><td>460</td><td>25%</td><td>97</td><td>42%</td></tr>
<tr><td>Zebra Crossing</td><td>163</td><td>9%</td><td>39</td><td>17%</td></tr>
<tr><td>Raised Zebra Crossing</td><td>1</td><td>0.1%</td><td>0</td><td>0%</td></tr>
<tr><td>Slip Lane Sub-total</td><td>741</td><td>41%</td><td>159</td><td>69%</td></tr>
<tr><td colspan="6"></td></tr>
<tr><td rowspan="4">Conventional Lane</td><td>Exclusive Lane</td><td>389</td><td>21%</td><td>27</td><td>12%</td></tr>
<tr><td>Shared lane</td><td>688</td><td>38%</td><td>44</td><td>19%</td></tr>
<tr><td>Conventional Lane Sub-total</td><td>1077</td><td>59%</td><td>71</td><td>31%</td></tr>
<tr><td>Total</td><td>1818</td><td>100%</td><td>230</td><td>100%</td></tr>
</table>

According to Table 4-9, exclusive left turn lanes experienced considerably fewer crashes than their share of treatments (12% versus 21%). Similarly, the shared left turn lanes experienced fewer crashes than their frequency in the signal network (19% versus 38%).

Ideally, traffic exposure using volumes should have been established for the intersections approaches of the various crashes. However, it was considered impracticable to do so for the overall crash analysis stage, due to the large sample size involved. Nevertheless, it is envisaged to consider the exposure factor in the detailed crash analysis as presented in Chapter 5.

It is worth noting that a significant number of rear end crashes were excluded from analysis due to lack of information presented in TCRs. Lacking information include the detailed location and/or description of the crash occurring at the intersection approach such as lane configurations, specifically at conventional lanes (and hence excluded from the analysis). Conversely, most of those crashes that occurred at slip lanes were indicated in the TCRs. Therefore, the author believes that rear end crashes that occurred at exclusive, shared left turn lanes were underrepresented in CAS. With this in mind, the overall crash frequency for both slip lanes and conventional lanes may be back to near parity situation.

4.3.3 Left turn crashes by treatments type and frequency –pedestrians only

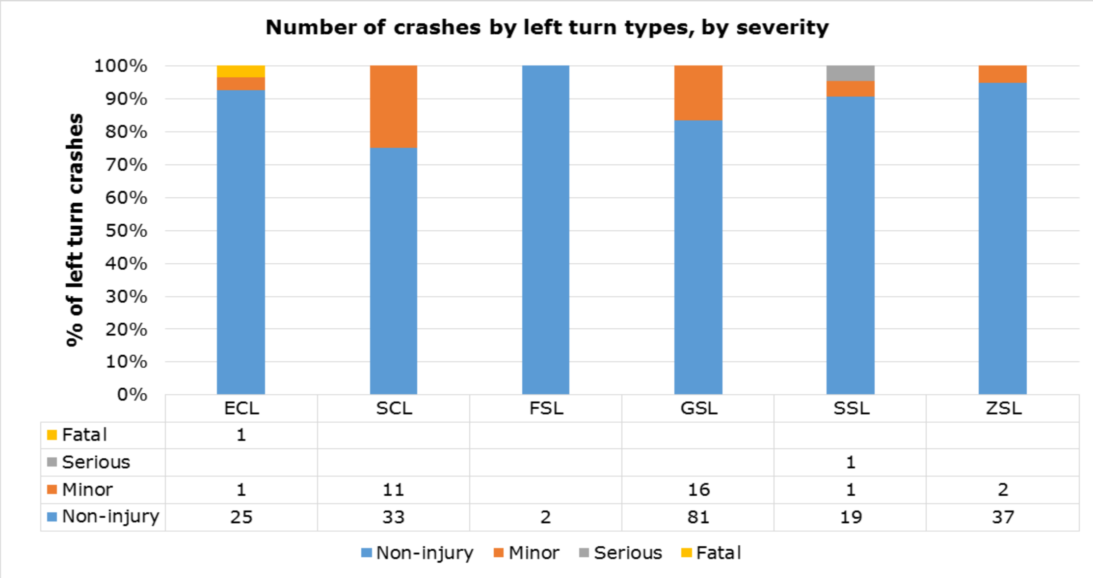
Table 4-10 Left turn pedestrian crashes, by type and treatment frequency in the network

Left Turn Treatment Type		Left turn treatments frequency in the network		Left turn pedestrian crashes	
		No. of left turn treatments in the signal network	% of left turn treatments in the signal network	No.	%
Slip Lane	Signalised	75	4%	0	0%
	Free Flow	42	2%	0	0%
	Give-way No Pedestrian Marking	460	25%	0	0%
	Zebra Crossing	163	9%	1	14%
	Raised Zebra Crossing	1	0.1%	0	0%
	Slip Lane Sub Total	**741**	**41%**	**1**	**14%**
Conventional Lane	Exclusive Lane	389	21%	0	0%
	Shared lane	688	38%	6	86%
	Conventional Lane Sub Total	**1077**	**59%**	**6**	**86%**
	Total	**1818**	**100%**	**7**	**100%**

Table 4-10 indicates that conventional lanes experienced 86% (7) of the crashes involving pedestrians while slip lanes experienced 14% (1) of the crashes. It appears that pedestrian crashes are not a major issue as there were only 7 crashes out of 230 crashes that involved pedestrians, which is considered insubstantial.

It is worth noting that most of the pedestrian crashes occurred at the shared conventional left turn lanes.

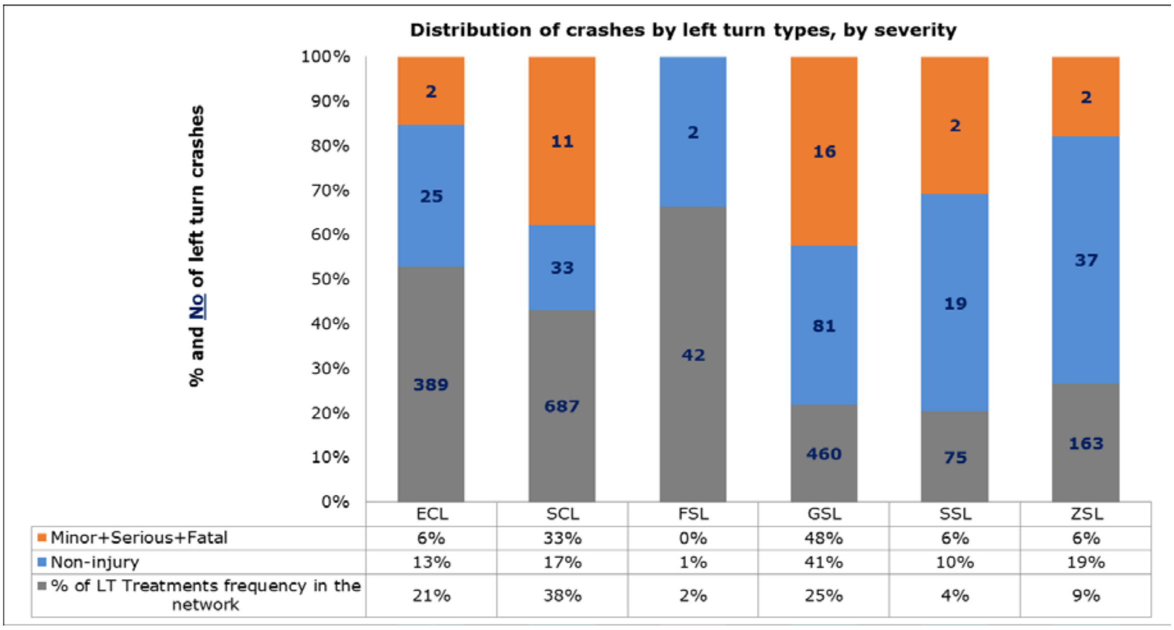
4.3.4 Left turn crashes by treatments type and severity



	ECL	SCL	FSL	GSL	SSL	ZSL
Fatal	1					
Serious					1	
Minor	1	11		16	1	2
Non-injury	25	33	2	81	19	37

Figure 4-10 Distribution of left turn crashes, by treatment types and severity

Figure 4-10 summarises the severity of left turn crashes for various treatments. It shows that the proportions of fatal and serious crashes were low for the majority of left turn treatments. In contrast, the majority of crash frequencies was non-injury for most of the left turn treatments, excepting for shared conventional lanes and give-way slip lanes where they experienced slightly a higher proportion of minor injury crashes: 11 and 16 respectively.



	ECL	SCL	FSL	GSL	SSL	ZSL
Minor+Serious+Fatal	6%	33%	0%	48%	6%	6%
Non-injury	13%	17%	1%	41%	10%	19%
% of LT Treatments frequency in the network	21%	38%	2%	25%	4%	9%

Figure 4-11 Distribution of left turn crashes, by treatment types and severity – combined injuries (F+S+M)

As depicted in Figure 4-11, give-way slip lanes experience a higher proportion of injury crashes than their frequency in the signal network (48% versus 25%). Shared conventional lanes appear to experience a greater proportion of injuries, but close to their frequency in the signal network. Exclusive conventional lanes have a lower proportion of injuries to their treatment frequency in the network (6% versus 21%).

Figure 4-12 illustrates that the proportion of injury crashes is slightly lower at slip lanes to their frequency in the signal network (61% versus 69%) while conventional lanes appear to experience higher occurrence of injury crashes to their treatments frequency (39% versus 31%).

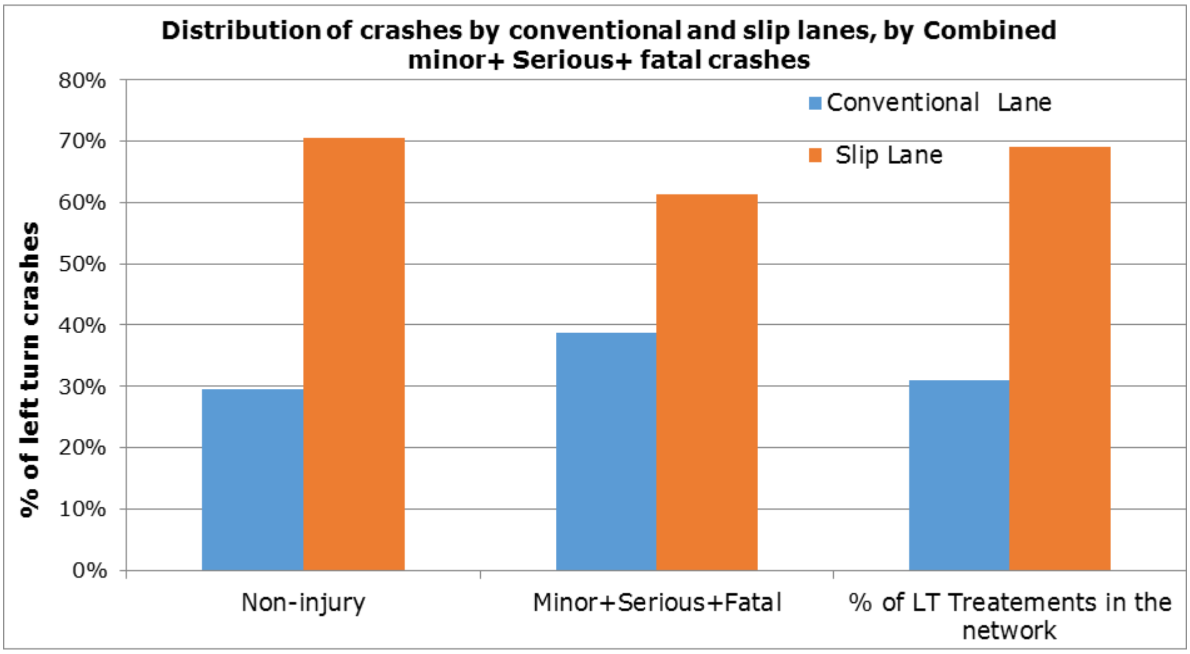
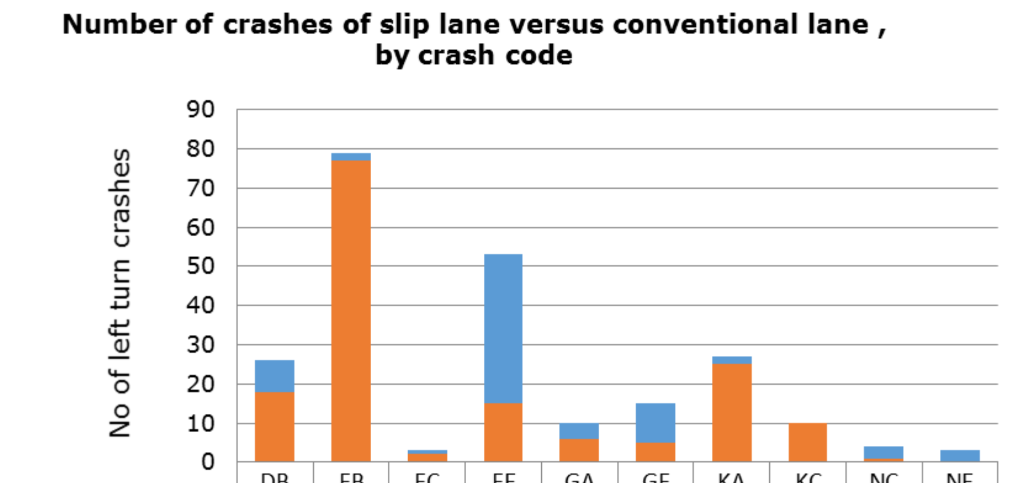


Figure 4-12 Distribution of combined crash severity for conventional lane versus slip lane

4.3.5 Left turn crashes by treatment type and crash code

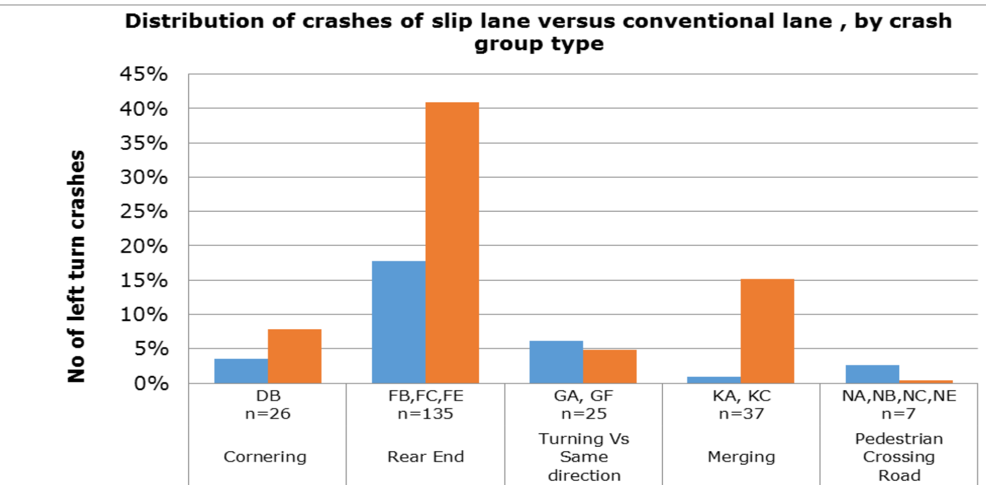
Figure 4-13 depicts the frequency of left turn crashes for conventional lanes in comparison to slip lanes by crash movement codes. It is clear that the majority of slip lanes crashes were FB type, while the FE type crashes were mainly associated with conventional lanes.



	DB	FB	FC	FE	GA	GF	KA	KC	NC	NE
Conventional Lane	8	2	1	38	4	10	2	0	3	3
Slip Lane	18	77	2	15	6	5	25	10	1	0

Figure 4-13 Distribution of crashes by movement code for conventional lane versus slip lanes

Figure 4-14 shows the proportion of left turn crashes for conventional lanes in comparison to slip lanes by classifying crashes into four groups. These groups are cornering, rear end, turning versus same direction, merging and pedestrian crashes. It is noticeably that the great proportion of crashes were rear end type for both of the two left turn treatments (59%). Slip lane and conventional lane have similar proportions of turning versus same direction crashes type (5% and 6% respectively). Merging related crashes were dominant for left turn slip lane.



	DB n=26 Cornering	FB,FC,FE n=135 Rear End	GA, GF n=25 Turning Vs Same direction	KA, KC n=37 Merging	NA,NB,NC,NE n=7 Pedestrian Crossing Road
Conventional Lane	3%	18%	6%	1%	3%
Slip Lane	8%	41%	5%	15%	0%

Figure 4-14 Distribution of crashes for conventional lane versus slip lane by crash group type

4.3.6 Left turn crashes by treatment type and contributing factors

As shown in Figure 4-15, there were two key contributing factors to left turn crashes for both conventional and slip lanes: in line of traffic and inattentive, and failed to notice.

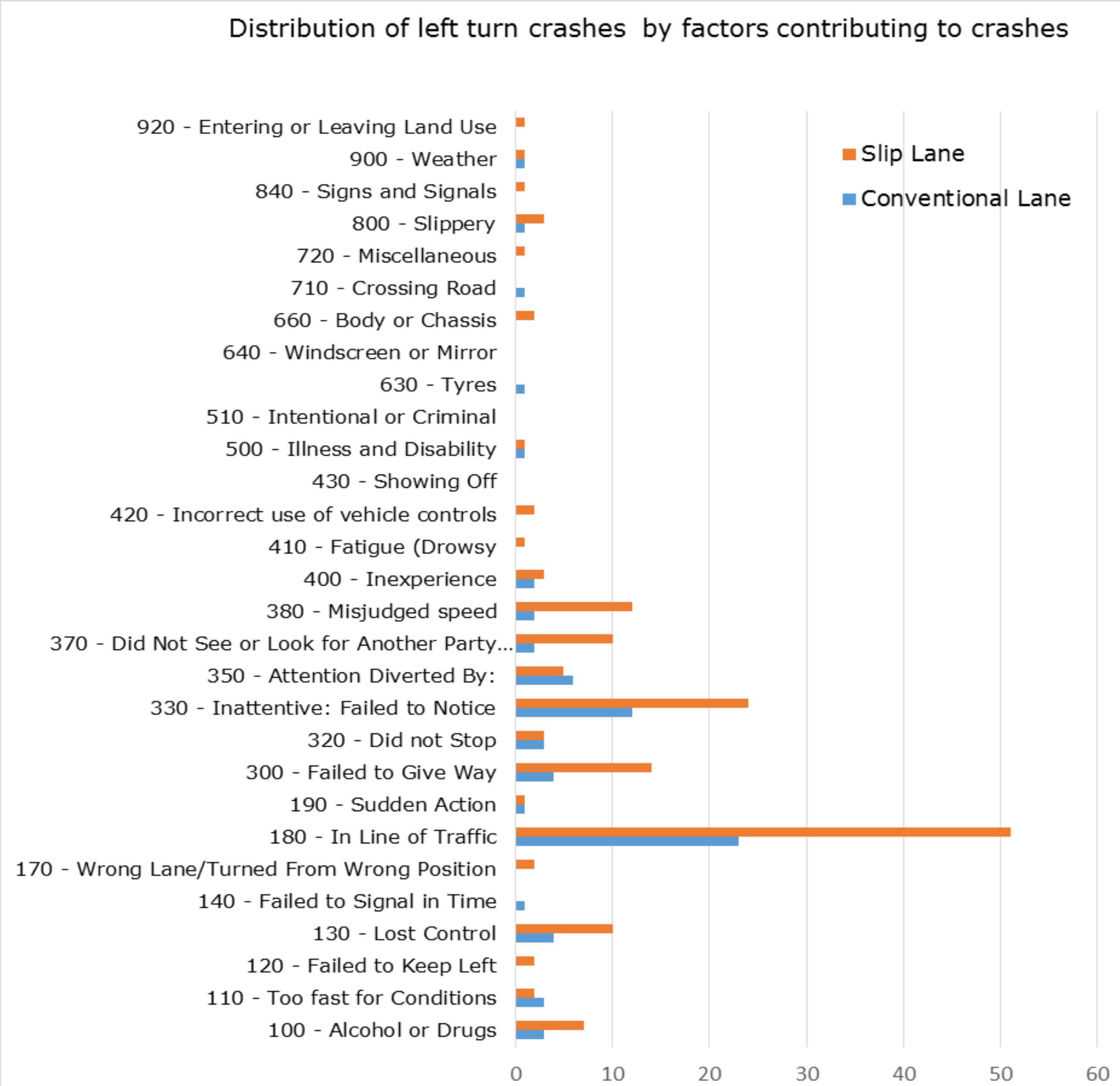


Figure 4-15 Distribution of crashes for conventional lane versus slip lane by contributing factors

4.4 Summary of overall crash analysis

The main aim of this analysis was to get a general understanding of the safety performance of the different left turn treatments type for the entire signals network.

A total of 230 left turn crashes were analysed using mainly two methods: unclassified left turn crashes and classified left turn crashes. In the unclassified left turn crash method, the crash data were analysed by: severity, movement types, contributing factors, light conditions, road surface conditions, month, day and time, sex and age group, vehicle type and object struck. On the other hand, in the second method, the crash data were analysed and classified into the various left turn treatment:

- Exclusive conventional lane;
- Shared conventional lane;
- Free flow slip lane;
- Give-way slip lane;
- Zebra crossing slip lane; and
- Signalised slip lane.

This analysis was conducted for all road users together and for pedestrians separately. Additionally, crashes were analysed by: severity, movement codes and contributing factors. The key findings of the two crash analysis methods is summarised in the following sections.

4.4.1 Unclassified left turn crash analysis

- The majority of left turn crashes were non-injury crashes (86%);
- The vast majority of crashes were rear-end/obstruction (70%);
- Pedestrian crashes only represented 3% (7) of the crash movement types;
- The two key contributing factors for left turn crashes were incorrect lane position (34%) and poor observation (24%);
- Comparison of both road and light conditions patterns of the left turn crashes with all signalised intersection crashes indicated that the proportion of crashes was similar in both cases. Most left turn crashes occurred in dry road (79%) and light (76%) conditions;
- The left turn crash frequencies by time of the day, day of the week and monthly were generally varied:
  - The majority of crashes occurred during PM peak (31%) time (3.30pm to 7 pm), followed by AM peak (28%);

  - The greatest proportion of crashes occurred on Friday (27%), while Sunday was the lowest (6%);
  - The highest percentage of crashes occurred in September and the least in April. The monthly proportional did vary, ranging from 9% to 28%.

- ❖ The sex and age group distribution of drivers involved in the left turn crashes showed minor patterns due to the small sample size. The non-injury crash data were not available which represented the largest number. However, few recognisable trends appeared to merge including:
  - Crash frequencies for male drivers aged 40 years and over remained the same with increasing age (15%), while female drivers generally decreased;
  - There were no female and male drivers in the same age group who had the same proportion of crashes;
  - The male and female drivers aged 30-39 years group appeared to have a higher rate of crash frequency (31% and 33%) compared to other age groups;
  - The male and female drivers experienced the same share of left turn crashes (13 versus 12).
- ❖ The greatest proportion of vehicle types that were involved in left turn crashes were cars; and
- ❖ The main three left turn crash frequency by object struck involved a post or pole (25%), traffic island (18%) and traffic sign (18%).

4.4.2 Classified left turn crash analysis

- ❖ Left turn slip lanes experienced 69% of the crashes at 41% of the sites;
- ❖ Left turn conventional lanes experienced 31% of the crashes at 59% of the sites;
- ❖ It appeared that pedestrian crashes are not a major issue as they represented 7 crashes out of 230 crashes which are insubstantial;
- ❖ The conventional shared left turn lane treatment accounted for 6 (86%) pedestrian crashes while the left turn slip lane with zebra crossing treatment accounted for only 1 (14%) pedestrian crash;
- ❖ Traffic exposure was considered impracticable for the overall crash analysis stage due to the large sample size involved. However, it is envisaged to consider the exposure factor in the next detailed crash analysis task;
- ❖ A substantial number of rear end crashes were excluded from analysis due to lack of information presented in TCRs such as lane configurations, specifically at conventional lanes. Therefore, the rear end crashes that

occurred at exclusive, shared left turn lanes were underrepresented in CAS and hence in the analysis;

- The great proportion of both slip lane and conventional lanes crashes were rear end type (41% and 18% respectively);
- The majority of slip lanes crashes were FB type (15), while the FE type (38) crashes were mainly associated with conventional lanes;
- The preponderance of crash frequencies was non-injury for most of the left turn treatments, excepting the shared conventional lanes and give-way slip lanes experienced a slightly higher proportion of minor injury crashes, 11 and 16 respectively;
- The give-way slip lane experienced a higher rate of injury crashes than their frequency in the signal network;
- Shared conventional lanes appeared to experience a greater proportion of injuries and close to their frequency in the signal network;
- Exclusive conventional lanes experienced a lower proportion of injuries to their treatment frequency in the network;
- The proportion of injury crashes was slightly lower at slip lanes to their frequency in the signal network (61% versus 69%) while conventional lanes appeared to experience a higher occurrence of injury crashes to their treatments frequency (39% versus 31%); and
- The two key contributing factors to left turn crashes for both conventional and slip lanes were in line of traffic and failed to notice.

5 Detailed Crash Analysis

This chapter documents the data collection, procedure and analysis performed, to carry out the detailed crash analysis task. The purpose of this analysis is to assess the safety performance of various left turn treatments for a selected sample of signalised intersections. Two major steps were involved in this analysis:

- ❖ Data collection and preliminary analysis; and
- ❖ Crash data analysis and results.

5.1 Data collection and preliminary analysis

The following three steps were undertaken to perform the detailed crash analysis:

- ❖ Site selection of the sample of signalised intersections, and gathering data for each site including left turn treatment type;
- ❖ Crash data and left turn crashes identification for selected sites; and
- ❖ Exposure data including traffic volume and pedestrian volume.

Each type of data is addressed in-depth in the following sections.

5.1.1 Intersection selection

A list of 84 signalised intersections has been selected from the main intersection data set that was produced in Chapter 3. These signalised intersections included a total of 267 approaches that allow left turns. The selected 267 approaches represent approximately 15% of total approaches on the entire signals network.

The following are the key selection criteria used when choosing the random sample of signalised intersections, for the detailed crash analysis task:

- ❖ Intersections selected have a mixture of approaches consisting of conventional left turn lanes and left turn slip lanes;
- ❖ Only three-leg and four-leg intersections were included. Multileg intersections were excluded because they typically incorporate unique features that are not representative of most intersections. Table 5-1 presents the location and type of the selected intersections;
- ❖ Intersections that have undergone a significant geometrical and/or operational change within the 5 year study period (2010 to 2014) were excluded;
- ❖ Intersections selected were located in a diverse geographical area within Auckland (north, south, east and west, as shown below), as well as a diversity in land-use environments including central business districts,

commercial and industrial areas, residential suburbs, and high-speed environments; and

❖ Intersections located on motorway off and on ramps were excluded.

Table 5-1 Type and location of selected intersection

Intersection Location (Area)	Cross–Intersections	Tee-Intersections	Total
North	7	6	13
South	16	10	26
Central	25	11	36
West	4	5	9
Total	**52**	**32**	**84**

A GIS map showing the geographic spread of the selected intersections from the signals network is indicated in Figure 5-1.

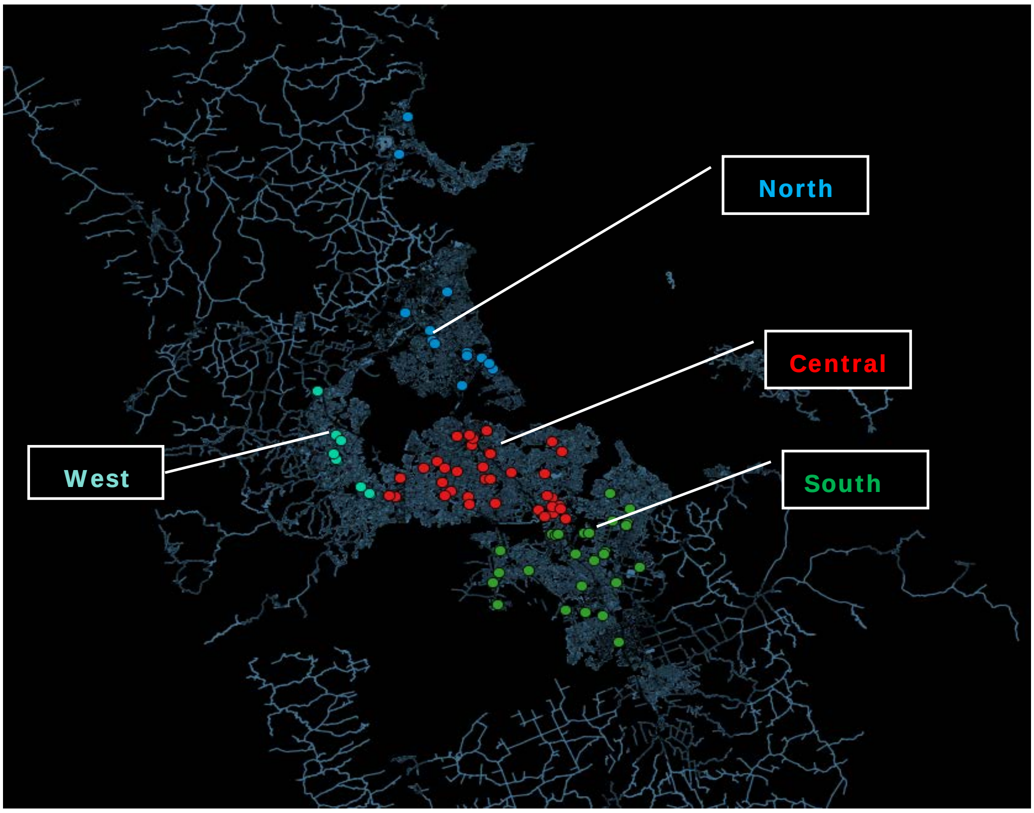


Figure 5-1 Selected intersection from north, south, central and west areas

A snapshot of the selected intersection approaches including left turn treatment classification is shown in Table 5-2, and the full list of selected sites are presented in Appendix D.

Table 5-2 A snapshot of the selected intersections approaches

AREA	Number	Description	North Approach	Exclusive	Shared	Giveway	Signalised	Free Flow	Zebra	Raised Zebra	South Approach	Exclusive	Shared	Giveway	Signalised	Free Flow	Zebra	Raised Zebra	East Approach	Exclusive	Shared	Giveway	Signalised	Free Flow	Zebra	Raised Zebra	West Approach	Exclusive	Shared	Giveway	Signalised	Free Flow	Zebra	Raised Zebra
North	1201	ANZAC ST - AUBURN ST	Auburn St	ECL							Auburn St				SSL				Anzac St	ECL							Anzac St		SCL					
North	1212	TAHAROTA RD - SMALES FARM ENTRANCE									TAHAROTO RD						ZSL										SMALES FARM						ZSL	
North	1216	NORTHCOTE RD - SMALES FARM - TAKAPUNA INT. SCH	The Avenue (Samles)						ZSL																		Northcote rd						ZSL	
North	1304	LAKE RD - JUTLAND RD - HAURAKI RD	Lake Rd						ZSL		Lake Rd		SCL						Hauraki Rd		SCL						Jutland Rd	ECL						
North	1305	LAKE RD - ESMONDE RD									Lake Rd				SSL				Esmonde Rd	ECL														
North	1402	GLENFIELD RD - MANUKU RD - HOGANS RD	GLENFIELD RD	ECL							GLENFIELD RD						ZSL		Hogans Rd	ECL							Manuku Rd	ECL						
North	1409	GLENFIELD RD - ALBANY HWY - SUNSET RD - GLENDHU	Albany Hwy	ECL							Albany Hwy		SCL						Sunset Rd						ZSL		Glendhu Rd						ZSL	
North	1413	GLENFIELD RD - CHIVALRY RD - MAYFIELD RD	GLENFIELD RD						ZSL		GLENFIELD RD		SCL						Chivalry Rd						ZSL		Mayfield Rd		SCL					
North	1607	EAST COAST RD - BROWNS BAY RD	EAST COAST RD						ZSL										Browns Bay Rd				SSL											
North	1617	ALBANY HWAY - OAKWAY DRIVE									Albany Hwy	ECL															Oakway Dr			GSL				
North	1807	SH1 (HIBISCUS COAST HWAY) / WEST HOE RD									HIBISCUS COAST HWAY	ECL															WEST HOE RD			GSL				
North	1810	SH1 (HIBISCUS COAST HWAY) / WHANGAPARAOA RD	HIBISCUS COAST HWAY			GSL					HIBISCUS COAST HWAY			GSL					WHANGAPARAOA RD					FSL			Millwater Parkway			GSL				
North	1942	SH1 - ONEWA RD - SYLVAN AVE									Sylvan Ave					FSL											Onewa Rd			GSL				
Central	2002	SYMONDS ST / KHYBER PASS RD / NEWTON RD	SYMONDS ST			GSL					SYMONDS ST	ECL							KHYBER PASS RD			GSL					NEWTON RD		SCL					
Central	2008	QUEEN ST / UPPER QUEEN ST / KARANGAHAPE RD	QUEEN ST						ZSL		UPPER QUEEN ST						ZSL		KARANGAHAPE RD		SCL						KARANGAHAPE RD	ECL						
Central	2013	GREEN LANE WEST / MANUKAU RD	MANUKAU RD		SCL						MANUKAU RD		SCL						GREEN LANE WEST						ZSL		GREEN LANE WEST	ECL						
Central	2051	PONSONBY RD / RICHMOND RD / PICTON ST	PONSONBY RD		SCL						PONSONBY RD		SCL						PICTON ST		SCL						RICHMOND RD	ECL						
Central	2086	GREEN LANE WEST / HOSPITAL GATE 3 / RACECOURSE	HOSPITAL GATE 3	ECL							RACECOURSE CAR PARK	ECL							GREEN LANE WEST		SCL						GREEN LANE WEST		SCL					
Central	2094	PITT ST / VINCENT ST	PITT ST						ZSL		PITT ST						ZSL		VINCENT ST		SCL						Hopetoun St		SCL					
Central	2109	ROSEBANK RD / ASH ST	ASH ST						ZSL		ASH ST			GSL					ROSEBANK RD		SCL						ROSEBANK RD		SCL					
Central	2128	KOHIMARAMA RD / KEPA RD									KOHIMARAMA RD				SSL												KEPA RD			GSL				
Central	2131	GILLIES AVE / EPSOM AVE	GILLIES AVE		SCL						GILLIES AVE		SCL						EPSOM AVE		SCL						EPSOM AVE		SCL					
Central	2145	MT ALBERT RD / MT EDEN RD / WARREN AVE	MT EDEN RD		SCL						WARREN AVE		SCL						MT ALBERT RD		SCL						MT ALBERT RD		SCL					
Central	2146	DOMINION RD / MT ALBERT RD	DOMINION RD		SCL						DOMINION RD		SCL						MT ALBERT RD		SCL						MT ALBERT RD		SCL					
Central	2147	HILLSBOROUGH RD / HERD RD / CARR RD	HILLSBOROUGH RD		SCL						HILLSBOROUGH RD		SCL						HERD RD		SCL						CARR RD		SCL					
Central	2153	NEW NORTH RD / MT ALBERT RD / CARRINGTON RD	CARRINGTON RD		SCL						MT ALBERT RD		SCL						NEW NORTH RD			GSL					NEW NORTH RD		SCL					
Central	2155	MT ALBERT RD / SANDRINGHAM RD	SANDRINGHAM RD		SCL						SANDRINGHAM RD		SCL						MT ALBERT RD			GSL					MT ALBERT RD			GSL				
Central	2159	MT SMART RD / ONEHUNGA MALL	ONEHUNGA MALL		SCL						ONEHUNGA MALL		SCL						MT SMART RD		SCL						MT SMART RD		SCL					
Central	2166	BROADWAY / REMUERA RD	BROADWAY	ECL															REMUERA RD						ZSL									
Central	2186	MT WELLINGTON HWAY / WAIPUNA RD / PENROSE RD	MT WELLINGTON HWAY		SCL						MT WELLINGTON HWAY						ZSL		WAIPUNA RD				SSL				PENROSE RD		SCL					
Central	2190	GREAT SOUTH RD / CHURCH ST EAST									GREAT SOUTH RD						ZSL										CHURCH ST EAST						ZSL	
Central	2193	MT WELLINGTON HWAY / SYLVIA PARK RD									MT WELLINGTON HWAY			GSL													SYLVIA PARK RD				SSL			
Central	2196	PENROSE RD / BARRACK RD	BARRACK RD		SCL																						PENROSE RD		SCL					
Central	2197	CARBINE RD / PANAMA RD	CARBINE RD	ECL																							PANAMA RD		SCL					
Central	2201	BALMORAL RD / DOMINION RD	DOMINION RD						ZSL		DOMINION RD						ZSL		BALMORAL RD						ZSL		BALMORAL RD						ZSL	
Central	2204	NEW NORTH RD / ST LUKES RD	ST LUKES RD		SCL						ST LUKES RD						ZSL		NEW NORTH RD			GSL					NEW NORTH RD						ZSL	
Central	2224	GREAT SOUTH RD / SYLVIA PARK RD	GREAT SOUTH RD					FSL											SYLVIA PARK RD			GSL												
Central	2228	ST LUKES RD / CORNWALLIS RD / KINGSWAY AVE	CORNWALLIS RD	ECL							KINGSWAY AVE		SCL						ST LUKES RD		SCL						ST LUKES RD						ZSL	
Central	2248	REMUERA RD / MIDDLETON RD	REMUERA RD		SCL														MIDDLETON RD	ECL														
Central	2259	SEART / CARBINE RD	CARBINE RD		SCL						CARBINE RD					FSL			South Eastern Hwy	ECL							South Eastern Hwy			GSL				
Central	2265	GREEN LANE EAST / LINK RD									Ellerslie Racecourse Dr				SSL				GREEN LANE EAST		SCL													
Central	2287	ST HELIERS BAY RD / APIRANA AVE									APIRANA AVE					FSL			ST HELIERS BAY RD			GSL												
Central	2307	MT WELLINGTON HWAY / SYLVIA PARK SHOPPING CEN	MT WELLINGTON HWAY						ZSL		MT WELLINGTON HWAY						ZSL		SYLVIA PARK SHOPPING CENTRE						ZSL		SYLVIA PARK SHOPPING CENTRE				SSL			
Central	2308	CARBINE RD / SYLVIA PARK BUSINESS CENTRE									CARBINE RD		SCL														SYLVIA PARK BUSINESS CENTRE						ZSL	
Central	2335	MAY RD / STODDARD RD / DENBIGH AVE	MAY RD		SCL						MAY RD	ECL							DENBIGH AVE		SCL						STODDARD RD		SCL					
Central	2342	ABBOTTS WAY / LUNN AVE / NGAHUE DR	NGAHUE DR	ECL							NGAHUE DR		SCL						LUNN AVE		SCL						ABBOTTS WAY	ECL						
Central	2354	CLARK ST / WARD ST	Ward St	ECL																							Clark St		SCL					
Central	2378	GREAT NORTH RD / MCCRAE WAY	Driveway		SCL						MCCRAE WAY		SCL						GREAT NORTH RD		SCL						GREAT NORTH RD		SCL					
Central	2905	PARNELL RISE / BEACH RD / STANLEY ST / THE STRAND	THE STRAND		SCL						STANLEY ST			GSL					PARNELL RISE	ECL							BEACH RD		SCL					
West	3009	GREAT NORTH RD / WEST COAST RD									West Coast Rd	ECL							Great North Rd						ZSL									
West	3010	GREAT NORTH RD / ARCHIBALD RD	Archibald Rd	ECL																							Great North Rd		SCL					
West	3011	GREAT NORTH RD / GLENVIEW RD / SABULITE RD	Sabulite Rd		SCL						Glenview Rd	ECL							Great North Rd		SCL						Great North Rd		SCL					
West	3017	GREAT NORTH RD / RAILSIDE AVE / RATANUI ST	Ratanui st	ECL							Railside Ave	ECL							Great North Rd		SCL						Great North Rd	ECL						
West	3018	GREAT NORTH RD / HENDERSON VALLEY RD / ALDERM	ALDERMAN DRV	ECL							Henderson Valley Rd	ECL							Great North Rd			GSL					Great North Rd			GSL				
West	3020	LINCOLN RD - SEL PEACOCK DRV	Lincoln Rd		SCL														Sel Peacook Dr		SCL													

5.1.2 Crash data collection

Crash data for the selected intersections were obtained from the main crash dataset produced in Chapter 3, for the five year period (2010-2014). The crash list has been interrogated by applying the same laborious process used in the overall crash analysis as presented in Chapter 4. After this process, a total of 389 out of 870 crash records remained. The remaining crash records were carried forward for the detailed crash analysis task.

A snapshot of the remaining crash lists and associated intersections with area, name and number for the selected intersections is shown in Table 5-3.

A macro within CAS crash query list was created for the 389 crash records. This macro was loaded back into CAS to acquire the TCRs for this list of crashes. There were in excess of 1500 pages obtained from CAS.

Table 5-3 A snapshot of the crash data for the selected intersections

AREA	Intersection	Description	CRASH ROAD	CRASH DIST	CRASH DIRN	INTSN	SIDE ROAD	CRASH ID	CRASH DATE	CRASH DOW	CRASH TIME	MVMT	VEHICLES	CAUSES	OBJECTS ST	ROAD CURVE	ROAD WET	LIGHT	WTHRa	JUNC TYPE	TRAF CTRL	ROAD MARK	SPD LIM	CRASH FATA	CRASH SEV	CRASH MIN	PERS AGE1	PERS AGE2
North	1607	EAST COAST RD - BROWNS B	BROWNS BAY ROAD			I	EAST COAST ROAD	201001223	27/01/2010	Wed	915	DB	PS1	135A 806		M	D	BF	F	T	T	R	50	0	0	1		
North	1305	LAKE RD - ESMONDE RD	ESMONDE ROAD			I	LAKE ROAD	201005031	1/08/2010	Sun	2219	FE	CE1C	109A		R	D	DO	F	T	T	C	50	0	0	1		
Central	2204	NEW NORTH RD / ST LUKES RD	NEW NORTH ROAD			I	ST LUKES ROAD	201034811	27/05/2010	Thu	1200	FB	CW1C	181A		R	W	BF	F	X	G	R	50	0	0	0		
North	1409	GLENFIELD RD - ALBANY HWY	GLENFIELD ROAD			I	GLENDHU ROAD	201036770	10/07/2010	Sat	1525	FE	CN1CC	108A 371A		R	D	BF	F	X	T	C	50	0	0	0		
West	3018	GREAT NORTH RD / HENDERS	GREAT NORTH ROAD			I	ALDERMAN DRIVE	201036899	14/07/2010	Wed	828	FB	CS1C	181A 387A		E	D	B	F	X	G	R	50	0	0	0		
North	1402	GLENFIELD RD - MANUKU RD -	MANUKA ROAD			I	GLENFIELD ROAD	201037880	4/08/2010	Wed	1832	FE	CE1C	181A 387A		R	D	DO	F	X	T	C	50	0	0	0		
Central	2186	MT WELLINGTON HWAY / WAIF	MOUNT WELLINGTON HIGHW			I	PENROSE ROAD	201038561	18/08/2010	Wed	840	DB	CN1	132A 137A	S	R	W	OF	FS	X	T	R	50	0	0	0		
South	4001	GEORGE BOLT MEM DR - TOM	GEORGE BOLT MEMORIAL DI			I	TOM PEARCE DRIVE	201041050	7/10/2010	Thu	1800	FB	CS1C	181A		R	D	BF	F	T	G	C	50	0	0	0		
South	4225	TI RAKAU DR - HARRIS RD	HARRIS ROAD	10	S		TI RAKAU DRIVE	201041924	11/11/2010	Thu	1710	FE	4N1C	129A		R	D	BF	F	T	T	C	50	0	0	0		
Central	2166	BROADWAY / REMUERA RD	REMUERA ROAD			I	BROADWAY	201042329	23/11/2010	Tue	850	FB	CW1C	331A		R	W	OF	F	T	G	C	50	0	0	0		
West	3009	GREAT NORTH RD / WEST COA	WEST COAST ROAD			I	GREAT NORTH ROA	201042360	29/11/2010	Mon	1030	FB	CN1C	330A 350A		R	D	BF	F	T	T	C	50	0	0	0		
North	1305	LAKE RD - ESMONDE RD	LAKE ROAD			I	ESMONDE ROAD	201042698	13/12/2010	Mon	933	FE	CN1C	181A		R	D	BF	F	T	T	L	50	0	0	0		
Central	2193	MT WELLINGTON HWAY / SYLV	SYLVIA PARK ROAD			I	MOUNT WELLINGTO	201042765	3/12/2010	Fri	1000	KA	TS2C	308B 925		R	D	BF	F	D	T	P	50	0	0	0		
West	3017	GREAT NORTH RD / RAILSIDE A	GREAT NORTH ROAD			I	RAILSIDE AVENUE	201043527	27/11/2010	Sat	2001	DB	CE2CC	386A	M	M	D	OF	F	X	T	R	50	0	0	0		
West	3009	GREAT NORTH RD / WEST COA	WEST COAST ROAD			I	GREAT NORTH ROA	201044133	12/12/2010	Sun	712	DB	VW2	131A	P	E	W	OF	L	T	G	C	50	0	0	0		
West	3010	GREAT NORTH RD / ARCHIBAL	GREAT NORTH ROAD			I	ARCHIBALD ROAD	201044135	25/12/2010	Sat	1509	FE	CE1C	181A		E	D	OO	F	T	T	C	50	0	0	0		
West	3017	GREAT NORTH RD / RAILSIDE A	GREAT NORTH ROAD			I	RAILSIDE AVENUE	201101286	7/02/2011	Mon	1455	FE	CW1C	512A		R	D	BF	F	X	T	C	50	0	0	1		
Central	2094	PITT ST / VINCENT ST	HOPETOUN ST			I	PITT ST	201101728	22/02/2011	Tue	1955	NC	CN2E	306A		M	D	BF	F	X	G	X	50	0	0	1	22	
West	3009	GREAT NORTH RD / WEST COA	WEST COAST ROAD			I	GREAT NORTH ROA	201102856	29/05/2011	Sun	58	FE	CE1C	181A		R	D	DO	F	T	T	R	50	0	0	1		
West	3009	GREAT NORTH RD / WEST COA	WEST COAST ROAD			I	GREAT NORTH ROA	201104952	13/10/2011	Thu	552	DB	CN2V	103A 111A 131A	IQ	M	W	DO	L	T	T	R	50	0	0	2		
North	1201	ANZAC ST - AUBURN ST	AUBURN ST			I	ANZAC ST	201105740	17/08/2011	Wed	1253	FE	4N1C	331A 363A 902		M	D	B	F	X	T	R	50	0	0	1		
West	3009	GREAT NORTH RD / WEST COA	WEST COAST ROAD			I	GREAT NORTH ROA	201130736	22/01/2011	Sat	2008	DB	4N1	111A 131A	P	M	D	OF	H	T	T	R	50	0	0	0		
West	3023	LINCOLN RD - UNIVERSAL DRV	UNIVERSAL DRIVE	5	W		LINCOLN ROAD	201131787	2/03/2011	Wed	1440	FE	4E1C	181A 387A		E	W	OF	L	X	T	C	50	0	0	0		
Central	2287	ST HELIERS BAY RD / APIRANA	ST HELIERS BAY ROAD			I	GLEN ATKINSON ST	201131874	1/03/2011	Tue	1815	KA	4E1TC	302B	Q	R	D	BF	F	T	G	C	50	0	0	0		
South	4608	TE IRIRANGI DR TOWN CENTRE	TE KOHA ROAD			I	TE IRIRANGI DRIVE	201132879	13/04/2011	Wed	250	DB	CN2	111A 131A	T	M	D	DO	F	X	T	R	80	0	0	0		
North	1942	SH1 - ONEWA RD - SYLVAN AV	ONEWA OFF NBD W			I	ONEWA ROAD	201134409	15/05/2011	Sun	1340	FB	VN1C	331A 353A 387A		R	W	OF	F	T	G	N	80	0	0	0		
Central	2049	REMUERA RD / VICTORIA AVE /	REMUERA ROAD			I	CLONBERN ROAD	201136902	28/06/2011	Tue	731	FE	CW1C	353A		R	W	BF	F	T	T	C	50	0	0	0		
Central	2186	MT WELLINGTON HWAY / WAIF	MOUNT WELLINGTON HIGHW			I	PENROSE ROAD	201138122	20/06/2011	Mon	1130	KC	CW2V	305A		R	W	O	H	X	T	R	50	0	0	0		
South	4225	TI RAKAU DR - HARRIS RD	HARRIS ROAD			I	TI RAKAU DRIVE	201140123	19/06/2011	Sun	453	DB	CW2	101A 111A 131A	PS	E	D	DO	F	T	T	R	50	0	0	0		
Central	2186	MT WELLINGTON HWAY / WAIF	WAIPUNA ROAD			I	MOUNT WELLINGTO	201140930	8/10/2011	Sat	945	FE	CS1C	101A		R	W	OF	L	X	T	C	50	0	0	0		
Central	2094	PITT ST / VINCENT ST	PITT ST			I	HOPETOUN ST	201143243	17/12/2011	Sat	550	GA	CN1C	181A		R	W	OO	M	X	G	C	50	0	0	0		
Central	2109	ROSEBANK RD / ASH ST	ASH ST			I	ROSEBANK ROAD	201201211	1/01/2012	Sun	2050	DB	CW1	101A 111A 131A	P	R	W	DO	H	X	T	C	50	0	0	2		
West	3018	GREAT NORTH RD / HENDERS	ALDERMAN DRIVE			I	GREAT NORTH ROA	201203068	2/04/2012	Mon	905	DB	CS2CT	111A 402A 414A		M	D	BF	F	X	G	C	50	0	0	1		
Central	2109	ROSEBANK RD / ASH ST	ASH ST			I	ROSEBANK ROAD	201203515	3/07/2012	Tue	910	NC	CN2E	306A 376A		R	W	OF	L	X	T	R	50	0	0	1	70	
Central	2259	SEART / CARBINE RD	SOUTH-EASTERN HIGHWAY			I	CARBINE ROAD	201233935	19/04/2012	Thu	1145	GF	TW1C	381A 671A		E	D	BF	F	X	T	R	50	0	0	0		
North	1305	LAKE RD - ESMONDE RD	ESMONDE ROAD			I	LAKE ROAD	201234109	14/01/2012	Sat	2111	DB	CN2	131A	FP	R	D	DO	F	T	G	C	50	0	0	0		
South	4330	HIGHBROOK DR - EL KOBAR D	HIGHBROOK DRIVE			I	EL KOBAR DRIVE	201234313	7/06/2012	Thu	540	GF	TN2V	179A		M	W	DO	F	T	T	C	50	0	0	0		
South	4001	GEORGE BOLT MEM DR - TOM	TOM PEARCE DRIVE			I	GEORGE BOLT MEM	201234890	4/04/2012	Wed	1645	FE	CN2C	181A		E	D	BF	F	X	T	C	50	0	0	0		
South	4126	ATKINSON AVE / PRINCES ST	PRINCES ST			I	ATKINSON AVENUE	201235591	31/05/2012	Thu	1658	KA	CW1B	302B		R	D	BF	F	X	G	X	50	0	0	0		
Central	2109	ROSEBANK RD / ASH ST	ROSEBANK ROAD			I	ASH ST	201236250	27/07/2012	Fri	1100	FE	CE1C	331A		R	D	BF	F	X	T	C	50	0	0	0		
South	4903	KIRKBRIDE RD - GEORGE BOL	20A/0/2.095			I	KIRKBRIDE ROAD	201237116	24/08/2012	Fri	1500	GA	CN1X	181A		E	D	BF	F	X	G	C	50	0	0	0		
West	3010	GREAT NORTH RD / ARCHIBAL	ARCHIBALD ROAD			I	GREAT NORTH ROA	201237836	23/07/2012	Mon	1100	FE	4S1C	331B 387B		R	W	O	M	T	T	C	50	0	0	0		
South	4126	ATKINSON AVE / PRINCES ST	PRINCES ST			I	ATKINSON AVENUE	201239037	24/08/2012	Fri	1530	GA	CE1C	181A		R	D	OF	F	X	T	C	50	0	0	0		
Central	2109	ROSEBANK RD / ASH ST	ROSEBANK ROAD			I	ASH ST	201240695	5/12/2012	Wed	815	FE	CN1C	181A		E		F		X	T	C	50	0	0	0		

5.1.2.1 Left turn crashes identification

Every crash record was manually interrogated, to identify the left turning related crashes. This was accomplished using the same technique that was used in the overall crash analysis task.

The results of the preliminary crash interrogation for the selected intersection approaches show that a total of 140 crashes were involved in left turning movements. The greater proportion of crashes did not involve left turning movements. Additionally, a negligible number of crashes did not have a TCR. These crashes were omitted from any additional analysis as indicated in Table 5-4.

Table 5-4 Preliminary crash results

Preliminary crash analysis	No of crashes	% of crashes
Crashes involving left turn movements	140	36%
Subtotal of crashes included	**140**	**36%**
Crashes not related to left turn movements	243	62%
TCR was not available	6	2%
Subtotal of crashes excluded	**249**	**64%**

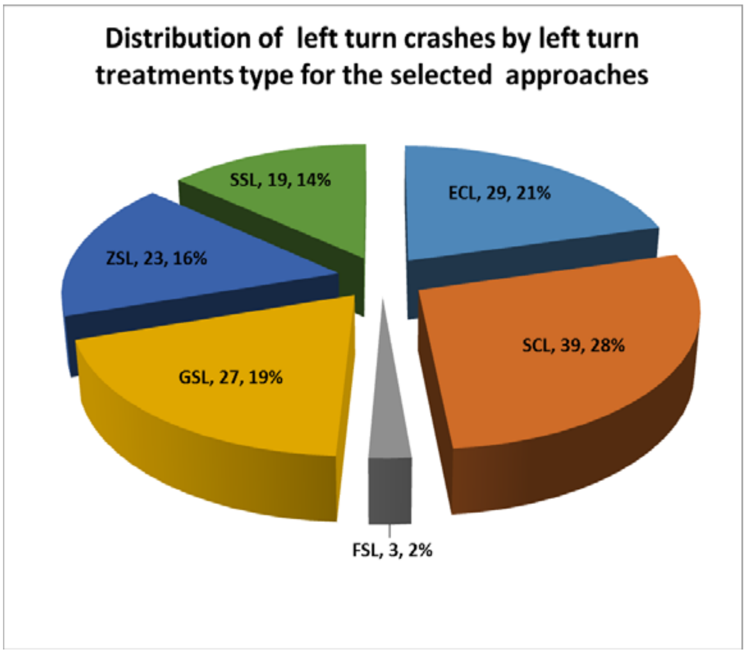
5.1.2.2 Left turn crash classification

Left turning crashes were classified according to the location of the crash by the left turn approach: weather at slip lane or conventional lane, and the type of facility at these lanes. As per the previous methodology used for the overall crash analysis, each crash record was assigned a code and inserted into the crash list spreadsheet. A snapshot of the spreadsheet is illustrated in Table 5-5.

Table 5-5 A snapshot of the crash list with assigned left turn code

AREA	intersection	Description	CRASH ROAD	CRASH DIST	CRASH DIRN	INTSN	SIDE ROAD	CRASH ID	CRASH DATE	CRASH DOW	CRASH TIME	MVMT	T Crash? SL/CL/N	VEHICLES	CAUSES	OBJECTS ST	ROAD CURVE	ROAD WET	LIGHT	WTHRa	JUNC TYPE	TRAF CTRL	ROAD MARK	SPD LIM	CRASH FATA	CRASH SEV	CRASH MIN	PERS AGE1	PERS AGE2
North	1201	ANZAC ST - AUBURN ST	AUBURN ST			I	ANZAC ST	201105740	17/08/2011	Wed	1253	FE	SSL	4N1C	331A 363A 902		M	D	B	F	X	T	R	50	0	0	1		
North	1204	TAHAROTO RD - NORTHCOTE RD	TAHAROTO ROAD			I	NORTHCOTE ROAD	201241111	21/11/2012	Wed	850	FB	ZSL	CN1C	181A 353A 387A		R	D	BF	F	X	G	P	50	0	0	0		
North	1204	TAHAROTO RD - NORTHCOTE RD	TAHAROTO ROAD			I	NORTHCOTE ROAD	201442122	21/08/2014	Thu	1741	FE	SCL	CS1C	181A		R	D	O	F	X	T	C	50	0	0	0		
North	1204	TAHAROTO RD - NORTHCOTE RD	TAHAROTO ROAD			I	NORTHCOTE ROAD	201442638	14/07/2014	Mon	1640	FB	ZSL	VN1V	353A 387A		R		BF	F	X	G	C	50	0	0	0		
North	1305	LAKE RD - ESMONDE RD	ESMONDE ROAD			I	LAKE ROAD	201005031	1/08/2010	Sun	2219	FE	ECL	CE1C	109A		R	D	DO	F	T	T	C	50	0	0	1		
North	1305	LAKE RD - ESMONDE RD	LAKE ROAD			I	ESMONDE ROAD	201042698	13/12/2010	Mon	933	FE	SSL	CN1C	181A		R	D	BF	F	T	T	L	50	0	0	0		
North	1305	LAKE RD - ESMONDE RD	ESMONDE ROAD			I	LAKE ROAD	201234109	14/01/2012	Sat	2111	DB	SSL	CN2	131A	FP	R	D	DO	F	T	G	C	50	0	0	0		
Central	1305	LAKE RD - ESMONDE RD	ESMONDE ROAD			I	LAKE ROAD	201444893	12/11/2014	Wed	1822	DB	SSL	VN2	111A 131A		M	D	OF	F	T	T	C	50	0	0	0		
North	1305	LAKE RD - ESMONDE RD	ESMONDE ROAD			I	LAKE ROAD	201447998	18/11/2014	Tue	2118	DB	SSL	VN2	111A 131A	IKS	E	D	DO	F	T	T	R	50	0	0	0		
North	1402	GLENFIELD RD - MANUKU RD - HOG	MANUKA ROAD			I	GLENFIELD ROAD	201037880	4/08/2010	Wed	1832	FE	ECL	CE1C	181A 387A		R	D	DO	F	X	T	C	50	0	0	0		
North	1409	GLENFIELD RD - ALBANY HWY - SU	GLENFIELD ROAD			I	GLENDHU ROAD	201036770	10/07/2010	Sat	1525	FE	SCL	CN1CC	108A 371A		R	D	BF	F	X	T	C	50	0	0	0		
North	1413	GLENFIELD RD - CHIVALRY RD - MA	CHIVALRY ROAD			I	GLENFIELD ROAD	201436341	28/03/2014	Fri	728	KC	ZSL	CS2C	302A 375A		E	D	OF	F	T	G	C	50	0	0	0		
North	1607	EAST COAST RD - BROWNS BAY R	BROWNS BAY ROAD			I	EAST COAST ROAD	201001223	27/01/2010	Wed	915	DB	ZSL	PS1	135A 806		M	D	BF	F	T	T	R	50	0	0	1		
North	1617	ALBANY HWAY - OAKWAY DRIVE	ALBANY HIGHWAY			I	OAKWAY DRIVE	201444335	26/09/2014	Fri	1222	GF	ECL	CN1C	172A 355A 372A 404A		E	D	O	F	T	T	C	50	0	0	0		
Central	1807	HIBISCUS COAST HWAY) / WEST HO	SCUS COAST HIGHWAY			I	WEST HOE ROAD	201441392	11/08/2014	Mon	1254	DB	ECL	TN1	129A 386A	KS	S	D	B	F	T	T	C	50	0	0	0		
North	1810	SH1 (HIBISCUS COAST HWAY) / WH	SILVERDALE PARKWAY			I	HIBISCUS COAST HIGH	201449859	19/12/2014	Fri	1320	FB	GSL	CE1C	181A		R	D	BF	F	X	G	C	50	0	0	0		
North	1942	SH1 - ONEWA RD - SYLVAN AVE	ONEWA OFF NBD W			I	ONEWA ROAD	201134409	15/05/2011	Sun	1340	FB	FSL	VN1C	331A 353A 387A		R	W	OF	F	T	G	N	80	0	0	0		
Central	2002	NDS ST / KHYBER PASS RD / NEWT	NEWTON ROAD			I	SYMONDS ST	201440902	4/08/2014	Mon	1300	FE	SCL	CE1C	350A 387A		R	D	BF	F	X	T	C	50	0	0	0		
Central	2008	ST / UPPER QUEEN ST / KARANGAH	QUEEN ST			I	KARANGAHAPE ROAD	201432119	28/02/2014	Fri	1500	FE	ZSL	CS1C	181A 330A		M	D	O	F	X	T	C	50	0	0	0		
Central	2008	ST / UPPER QUEEN ST / KARANGAH	KARANGAHAPE ROAD			I	QUEEN ST	201442626	10/07/2014	Thu	1155	FE	ECL	CE1C	181A		R	W	OF	F	X	T	C	50	0	0	0		
South	2008	ST / UPPER QUEEN ST / KARANGAH	KARANGAHAPE ROAD			I	UPPER QUEEN ST	201445845	14/10/2014	Tue	2140	KC	ZSL	VN2C	129A 386A 129B 386B		E	D	DO	F	X	G	L	50	0	0	0		
South	2008	ST / UPPER QUEEN ST / KARANGAH	KARANGAHAPE ROAD			I	UPPER QUEEN ST	201448954	1/12/2014	Mon	1800	KC	ZSL	BN2C	315B		E	D	B	F	X	G	C	50	0	0	0		
Central	2013	GREEN LANE WEST / MANUKAU RD	MANUKAU ROAD			I	GREEN LANE WEST	201410179	22/01/2014	Wed	1510	GA	SCL	CS1C	181A 331A		R	D	B	F	X	T	C	50	0	0	1		
Central	2013	GREEN LANE WEST / MANUKAU RD	MANUKAU ROAD			I	GREEN LANE WEST	201441187	27/06/2014	Fri	1700	GF	ZSL	MW2C	381A		R	D	BF	F	X	T	C	60	0	0	0		
Central	2049	REMUERA RD / VICTORIA AVE / CLO	REMUERA ROAD			I	CLONBERN ROAD	201136902	28/06/2011	Tue	731	FE	SCL	CW1C	353A		R	W	BF	F	T	T	C	50	0	0	0		
Central	2049	REMUERA RD / VICTORIA AVE / CLO	VICTORIA AVENUE			I	REMUERA ROAD	201435237	13/03/2014	Thu	900	DB	ECL	BS1	129A 386A	S	R	D	O	F	T	T	C	50	0	0	0		
Central	2051	NSONBY RD / RICHMOND RD / PICTON	PONSONBY ROAD			I	RICHMOND ROAD	201440674	17/07/2014	Thu	1024	GF	ECL	TE2C	172A 370A		R	D	B	F	X	T	C	40	0	0	0		
Central	2094	PITT ST / VINCENT ST	HOPETOUN ST			I	PITT ST	201101728	22/02/2011	Tue	1955	NC	ZSL	CN2E	306A		M	D	BF	F	X	G	X	50	0	0	1	22	
Central	2094	PITT ST / VINCENT ST	PITT ST			I	HOPETOUN ST	201143243	17/12/2011	Sat	550	GA	ZSL	CN1C	181A		R	W	OO	M	X	G	C	50	0	0	0		
Central	2094	PITT ST / VINCENT ST	HOPETOUN ST			I	PITT ST	201301907	12/03/2013	Tue	1022	FE	SCL	CE1M	331A 350A		R	D	BF	F	X	T	C	50	0	0	1		
Central	2094	PITT ST / VINCENT ST	PITT ST			I	VINCENT ST	201438433	10/06/2014	Tue	803	NC	SCL	BS2E	323A 334A 901		M	W	O	H	X	T	C	50	0	0	0		
Central	2109	ROSEBANK RD / ASH ST	ASH ST			I	ROSEBANK ROAD	201201211	1/01/2012	Sun	2050	DB	ZSL	CW1	101A 111A 131A	P	R	W	DO	H	X	T	C	50	0	0	2		
Central	2109	ROSEBANK RD / ASH ST	ASH ST			I	ROSEBANK ROAD	201203515	3/07/2012	Tue	910	NC	SCL	CN2E	306A 376A		R	W	OF	L	X	T	R	50	0	0	1	70	
Central	2109	ROSEBANK RD / ASH ST	ROSEBANK ROAD			I	ASH ST	201236250	27/07/2012	Fri	1100	FE	SCL	CE1C	331A		R	D	BF	F	X	T	C	50	0	0	0		
Central	2109	ROSEBANK RD / ASH ST	ROSEBANK ROAD			I	ASH ST	201240695	5/12/2012	Wed	815	FE	SCL	CN1C	181A		E		F		X	T	C	50	0	0	0		
Central	2109	ROSEBANK RD / ASH ST	ROSEBANK ROAD			I	ASH ST	201301933	14/03/2013	Thu	1145	FE	SCL	4S1C	181A		R	D	BF	F	X	T	C	50	0	0	1		
Central	2109	ROSEBANK RD / ASH ST	ROSEBANK ROAD	10	S		ASH ST	201333794	16/05/2013	Thu	1645	FE	SCL	CN1C	181A		R		F		X	T	C	50	0	0	0		
Central	2109	ROSEBANK RD / ASH ST	ROSEBANK ROAD			I	ASH ST	201440462	26/07/2014	Sat	1725	FE	ECL	CS1C	181A 330A		R	D	B	F	M	T	L	50	0	0	0		
South	2131	GILLIES AVE / EPSOM AVE	GILLIES AVENUE			I	EPSOM AVENUE	201416764	24/10/2014	Fri	1700	NE	SCL	CW2E	307A 330A		R	D	B	FS	X	T	C	50	0	0	1	28	
Central	2145	LBERT RD / MT EDEN RD / WARREN	OUNT ALBERT ROAD			I	WARREN AVENUE	201437165	6/05/2014	Tue	1932	FE	SCL	CN2C	181A		R	W	OF	L	X	T	C	50	0	0	0		
Central	2146	DOMINION RD / MT ALBERT RD	DOMINION ROAD			I	MOUNT ALBERT ROAD	201433763	10/02/2014	Mon	1100	FE	SCL	CN1C	181A		R	D	B	F	X	T	C	50	0	0	0		
Central	2147	LSBOROUGH RD / HERD RD / CARR	ILLSBOROUGH ROAD			I	CARR ROAD	201430923	28/01/2014	Tue	1935	FE	SCL	CN1C	181A		R	D	BF	F	T	T	C	50	0	0	0		
Central	2147	LSBOROUGH RD / HERD RD / CARR	ILLSBOROUGH ROAD			I	CARR ROAD	201438938	25/06/2014	Wed	1720	FE	SCL	CN1C	331A 387A		R	W	DO	L	T	T	C	50	0	0	0		
Central	2153	RTH RD / MT ALBERT RD / CARRING	OUNT ALBERT ROAD			I	NEW NORTH ROAD	201435345	9/04/2014	Wed	1510	FE	SCL	CW1C	181A		R	D	B	F	X	T	C	50	0	0	0		
Central	2153	RTH RD / MT ALBERT RD / CARRING	NEW NORTH ROAD	5	W		MOUNT ALBERT ROAD	201441114	5/08/2014	Tue	1530	FC	SCL	TS1C	331A		R	D	BF	F	X	T	C	50	0	0	0		

Figure 5-2 shows the proportion of left turn crashes by left treatment type and code for all road users.



Code	Left Turn Type
ECL	Exclusive Conventional Lane
SCL	Shared Conventional Lane
FSL	Free Slip Lane
GSL	Giveway Slip Lane
ZSL	Zebra Slip Lane
SSL	Signalised Slip Lane

Figure 5-2 Distribution of left turn crashes by left turn types for the selected samples

Figure 5-2 indicates the following:

- ❖ The greatest proportion of left turn crashes occurred at shared conventional lanes, followed by exclusive left turn lanes and then give-way slip lanes; and
- ❖ The lowest proportion of left turn crashes occurred at the free slip lanes.

Table 5-6 indicates the number of left turn crashes that involved pedestrians occurring at a particular site from the selected intersections.

Table 5-6 Number of pedestrian crashes

Intersection No	No of pedestrian crashes
2094	2
2109	1
2201	1
2378	1
4137	1
Total	**6**

5.1.3 Exposure data collection

Three types of exposure data were considered for collection and analysis:

- ❖ Frequency data of the left turn treatments;
- ❖ Traffic flow data of the left turn treatments; and
- ❖ Pedestrian count data.

5.1.3.1 Frequency exposure of left turn treatments

Without the need for pedestrian and traffic flow data, this method is a realistic measure of the frequency of each left turn treatment at the selected signalised intersections. This method has been used in the overall crash analysis which is similar to what O'Brien et al. (2010) adapted in their research study.

This method involved classifying the selected intersection approaches into the various left turn treatment types. Later on in the crash analysis, the left turn crashes corresponding to each treatment type will be compared with their relevant left turn frequency.

The proportion of left turn treatments is presented in Table 5-7.

Table 5-7 Summary of selected intersections approaches and left turn treatments

Left turn treatment type For selected intersections approaches	Proportion of the treatments in the signals network	
	Number	Percent
Left Turn Slip Lane		
Signalised Control	17	6%
Free Flow	14	5%
Give-way No Pedestrian Marking	57	21%
Zebra Crossing	38	14%
Raised Zebra Crossing	0	0%
Slip Lane Total	**126**	**47%**
Conventional Left Turn Lane		
Exclusive Lane	41	15%
Shared lane	100	37%
Conventional Lane Total	**141**	**53%**
Total no of LT approaches	**267**	**100%**

This method has a limitation as it is subject to some bias, in terms of installation of a particular type of left turn treatment at locations where traffic conditions are heavy compared to others. For example, a higher proportion of the crashes may be related to the high traffic volume characteristic of these sites compared with other left turn treatments. Therefore, another exposure measure which is based on traffic flow was considered.

Another way to examine the results was the frequency exposure method. However, the frequency exposure method, gives a reasonable and relative measure of the safety performance of various left turn treatments without taking into account traffic exposure effects.

Figure 5-3 depicts the distribution of left turn slip lanes compared with left turn conventional lanes for left turn treatments types for the selected approaches. It can be seen that the proportion of left turn slip lanes was slightly lower (6%) in comparison to the conventional lanes.

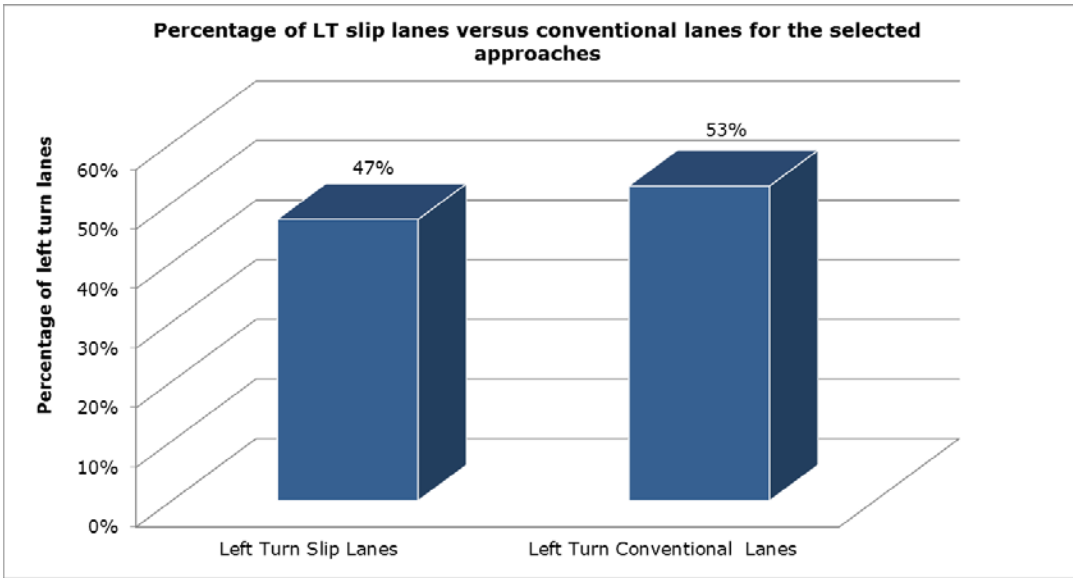


Figure 5-3 Distribution of the left turn slip lanes versus conventional lanes for the selected approaches

5.1.3.2 Traffic flow data exposure

5.1.3.2.1 Influence of traffic flow

In order to assess the safety performance of various left turn treatments, the influence of traffic volume on crashes was an essential consideration. The omission of traffic flow data was specified as a significant gap in the analysis of previous studies on the safety analysis of various left turn treatments.

Most of the studies indicate that there is certainly a relationship between the number of crashes and traffic volume. Consequently, the effect of traffic volume on crashes was necessary to answer the following question:

Are the left turn slip lanes experiencing more crashes than the conventional lanes in comparison to traffic volumes and vice versa?

Thus, traffic counts were required to calculate a crash rate based on traffic volume exposure. Crash rate can be an effective tool in measuring the relative safety of a particular left turn treatment. The crash rate is measured as a product of the conflicting traffic flows entering from each approach at an intersection. However, this method was not applicable to the present study for the following reasons:

- ❖ It includes the whole intersection; nevertheless, this study focuses on each approach individually;

- There are movements that could be in conflict with a particular type of left turn treatment but not for other treatments. For example, at a signalised slip lane, the left turn movements do not conflict with the right turn movements. However, at a give-way slip lane, this conflict exists;
- Even if the crash exposure equations were able to be used as in crash prediction models, then the results of crash data would need to be disaggregated. While data could be disaggregated in this way, the resulting crash frequencies at any given left turn treatment would have been so low as to prevent a solid conclusion; and
- It requires turning volumes in order to estimate exposure for certain movements and such data was not available for the present study.

For the purpose of this research, a simpler method of calculating crash rate was adapted in this study by measuring the traffic flows entering each approach.

5.1.3.2.2 Traffic counts

The traffic flow data was extracted from SCATS® detector loop count data for a month period over 24 hours, during March 2016. This data was extracted from SCATS® in the form of text file outputs for each site. This text file contains the vehicles that go over individual detector loops for each site.

SCATS® detector loop counts for each site were imported into an Excel spreadsheet for data manipulation. Vehicle counts were averaged for the month period for each detector loop. A SCATS® picture was used to identify detectors located in each lane approach. For each site, the average volume from individual loops was summed together for each approach. This was to establish the total entry volumes for each left turn approach treatment as indicated in Figure 5-4.

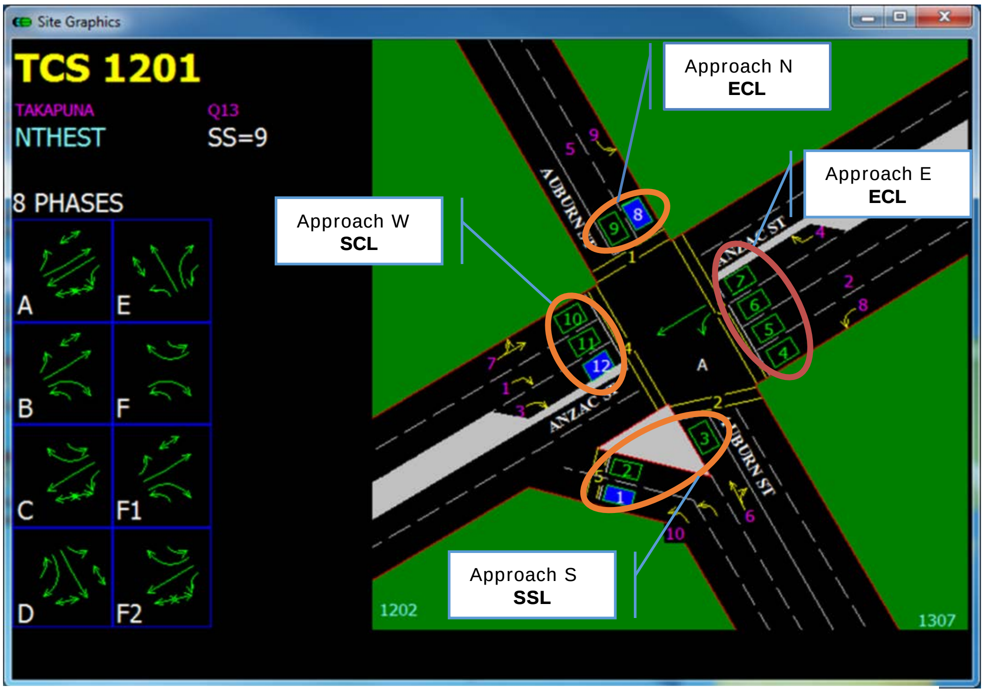


Figure 5-4 SCATS® picture indicates detector loops in each approach

During this period, it was found that some detector loops were faulty and showing bad data errors. The data from these erroneous detector counts was removed and replaced with data from other days.

Manual turning traffic count data were not available for the present study. Therefore, the SCATS® loop counts were used instead. However, SCATS® detector loop counts do not provide separate turning counts if it is located within a shared movement such as a shared through and left-turn lane. Additionally, SCATS® loop detectors were not always present at all left turn slip lanes except signalised slip lanes. Since the purpose of the traffic volumes were solely to identify which approach is busy in comparison with others, it was considered an adequate measure.

For this research, the purpose of the traffic volume was solely to identify which approach is busy in comparison with others. Hence, the traffic volume at selected intersections was aggregated by approach level instead of by individual left turn movements, which was considered an adequate measure.

SCATS® detector loop counts (for a month) for a sample site is presented in Appendix E.

5.1.3.2.3 Traffic counts analysis

Figure 5-5 shows the distribution of the daily traffic volume entering for all approaches (vpd) for the selected sample approaches. The sample set of approaches show the volume varies in range from less than 1,000 to more than 20,000 vpd.

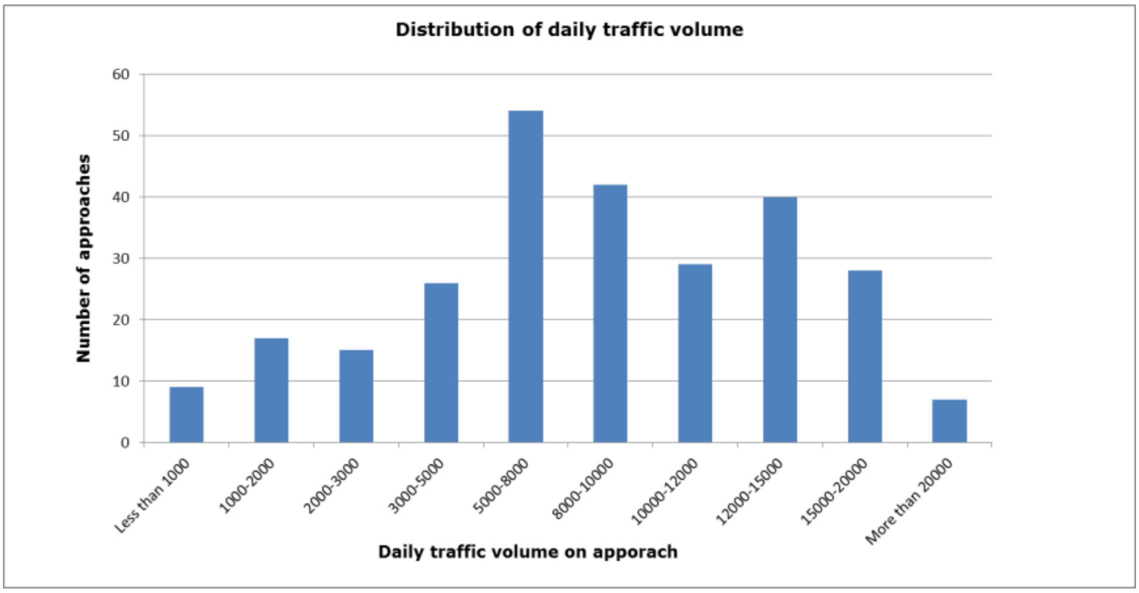


Figure 5-5 Distribution of daily traffic volume (ADT)

The distribution of the average daily traffic volume for each left turn treatment is shown in Figure 5-6. In contrast, the distribution of the average daily traffic volume for conventional lanes versus slip lanes is presented in Figure 5-7.

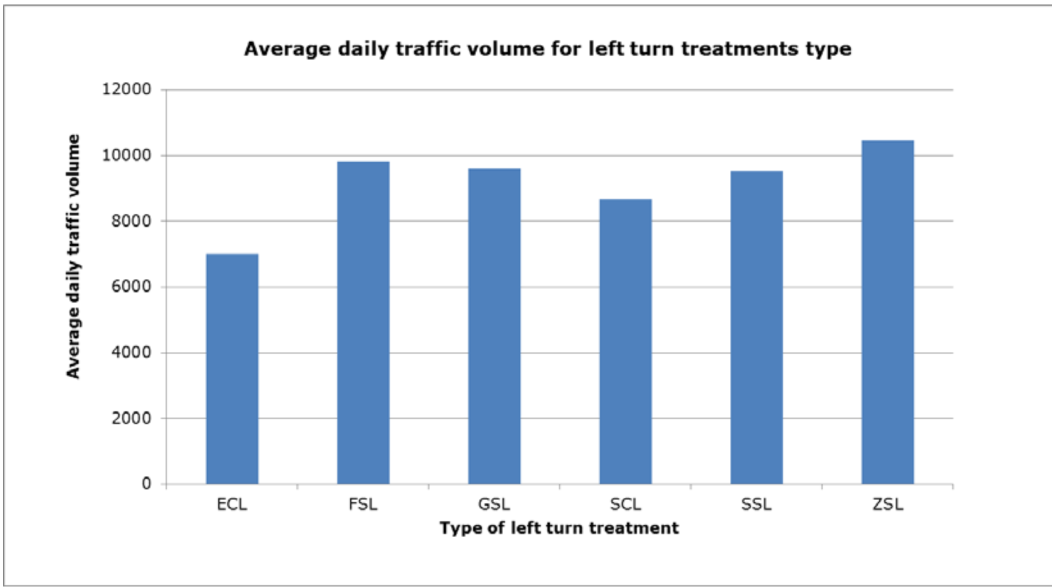


Figure 5-6 Distribution of average daily traffic volume for each left turn approach type

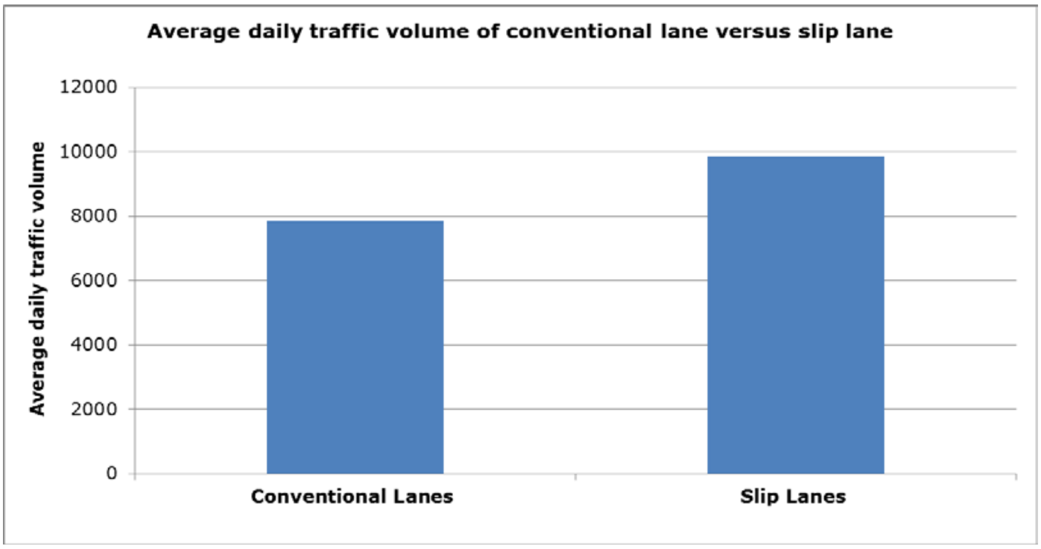


Figure 5-7 Distribution of average daily traffic volume for conventional lanes against slip lanes approaches

From Figure 5-6 and Figure 5-7 the key findings are:

- The shared lanes form the greatest proportion of traffic volume for the conventional lane, followed by the zebra crossing lanes for slip lane; and
- The left turn slip lanes formed the largest traffic volume even if the volume of some slip lanes were not available as indicated before.

5.1.3.3 Pedestrian counts data exposure

Data on the number of pedestrian phases activated per day (pedestrian calls) were extracted from SCATS. The pedestrian activations do not represent the actual number of crossing pedestrians; however, it can be used as a proxy measure on relative levels of pedestrian usage facing each of the left turn treatment types as indicated in Table 5-8.

Therefore, it can be used as a substitute for the pedestrian volume counts. Sites where pedestrian crashes occurred were considered for data collection only (pedestrian calls). These sites numbers are 2094,2109,2201,2378, and 4137.

Table 5-8 Pedestrian usage level corresponds to number of crossings

Pedestrian usage level	No of pedestrian crossing legs
Low level-pedestrian calls ranged from 0 to 120	9
Medium level-pedestrian calls ranged from 120 to 220	6
High level-pedestrian calls over 220	5

5.2 Crash data analysis and results

Three methods were used to analyse left turn crashes at various left turn treatments for the selected sample of signalised intersections, applying different exposure measures:

- ❖ Analyse crash data by frequency exposure;
- ❖ Analyse crash data by traffic volume exposure; and
- ❖ Analyse pedestrian crash data by pedestrian activation exposure.

The detail of the three crash analysis methods are described in the following sections.

5.2.1 Crash data by frequency exposure

The crash data resulting from the investigation and classification is presented in Table 5-9. The crash data included all road user crashes. These data split down the crashes for the selected sample into the various types of left turn slip lanes and left turn conventional lanes. In addition, data including left turn treatment frequency and crash rate were included to estimate the relevant safety performance of various left turn treatments.

Table 5-9 Left turn crashes and treatment frequency in the selected samples by the type of left turn

Left Turn Treatment Type		Left turn treatments frequency Exposure for selected samples		Left turn crash types		Left turn crash rate (5Yrs)
		No. of left turn treatments	% of left turn treatments	No. of left turn crashes	% of left turn crashes	
Slip Lane	Signalised	17	6%	19	14%	1.118
	Free Flow	14	5%	3	2%	0.214
	Giveway No Pedestrian Marking	57	21%	27	19%	0.474
	Zebra Crossing	38	14%	23	16%	0.605
	Raised Zebra Crossing	0	0%	0	0%	0.000
	Slip Lane Sub Total	**126**	**47%**	**72**	**51%**	**0.571**
Conventional Lane	Exclusive Lane	41	15%	29	21%	0.707
	Shared lane	100	37%	39	28%	0.390
	Conventional Lane Sub Total	**141**	**53%**	**68**	**49%**	**0.482**
	Overall	**267**	**100%**	**140**	**100%**	**0.524**

The key findings:

Slip lanes

- ❖ Signalised slip lanes accounted for 14% of crashes at 6% of the selected samples. Assuming the traffic flows to be equal at these samples, signalised slip lanes experience higher crash rate than their frequency. This was proven to be statistically significant;
- ❖ The give-way slip lanes experienced 19% of crashes which is slightly lower than their frequency by 2%; however, the statistical significance has not been demonstrated;

- The zebra crossing slip lanes appeared to experience 2% higher occurrence of crashes compared to their frequency (16% versus 14%). Once more this statistically significance has not been demonstrated;
- The free flow slip lane treatments have slightly lower occurrence of crashes to their frequency (2%, 5%), but the statistical significance has not been demonstrated; and
- The left turn slip lanes experienced a slightly higher occurrence of crashes to their relevant frequency in the selected samples (51% versus 47%).

Conventional lanes

- Shared lane treatments have experienced 9% lower occurrence of crashes to their frequency, which is contrary to expectations (28% versus 37%). This result agrees with the overall crash analysis. This may be due to the issue of crash data underrepresentation at shared lanes as explained in Chapter 4. The exclusive lane treatments shared 21% of the crashes while comprising 15% of their frequency. However, the statistical significance of these results has not been demonstrated; and
- The left turn conventional lanes experienced a slightly lower occurrence of crashes compared to their relative treatment frequency in the selected samples (49% versus 53%).

In summary, the left turn crash rate for slip lanes was slightly higher than conventional lanes, nevertheless, it was more likely to be similar (0.48 versus 0.57). Therefore, their safety performance is similar, which was statistically proven, with some notable differences according to the treatment type.

5.2.2 Crash data by traffic volume exposure

The combination of crash frequency and vehicle exposure results in the crash rate. Crash rates are expressed as crashes per Million Entering Vehicles (MEV) for each type of left turn treatment approaches.

Crash Rate (CR) = Crash Frequency / Exposure;

Crash Frequency = total number of crashes for 5 years for each type of left turn treatment;

$Expsoure = ADTx\ 365x\ No.YRS(5)/100{,}000{,}000\ where;$

ADT= average daily count for each type of left turn treatment approaches;

YRS=period of study which is 5 years; and

CR is expressed as a crash per Million entering vehicles per left turn treatment approaches.

Table 5-10 displays the distribution of left turn crashes by type of treatments, exposure, and crash rate.

Table 5-10 Left turn crashes, exposure and crash rate for selected samples

Left Turn Treatment Type		Exposure per entering vehicles X 10^8 X 5Yrs (100 MEV)		Left turn crash types		Left turn crash rate (5Yrs)
		approach Volume	% of approach Volume	No. of left turn crashes	% of left turn crashes	
Slip Lane	Signalised	3.0	7%	19	14%	6.42
	Free Flow	2.5	6%	3	2%	1.19
	Giveway No Pedestrian Marking	10.0	23%	27	19%	2.70
	Zebra Crossing	7.2	17%	23	16%	3.18
	Raised Zebra Crossing	0.0	0%	0	0%	0.00
	Slip Lane Sub Total	**22.7**	52%	**72**	**51%**	**3.17**
Conventional Lane	Exclusive Lane	5.2	12%	29	21%	5.53
	Shared lane	15.8	36%	39	28%	2.46
	Conventional Lane Sub Total	**21.1**	48%	**68**	**49%**	**3.23**
	Overall	**43.8**	**100%**	**140**	**100%**	**3.20**

Key findings:

Slip lanes

- Give-way slip lane accounted for 19% of crashes but 4% lower than their traffic volume exposure (19% < 23%); which is similar to their frequency exposure results;
- Signalised slip lanes experienced a considerable proportion of crashes (14%) compared to their traffic volume exposure (7%), which correspond with the overall crash analysis (9% crashes versus 4% frequency) and frequency exposure analysis;
- Zebra crossing slip lanes appeared to have 1% lower occurrence of crashes compared to their traffic volume exposure with 16% of crashes and 17% exposure. This result does not agree with overall crash analysis or frequency exposure analysis; and
- Free flow slip lanes accounted for lowest crash proportions compared to other slip lane treatments. Moreover, it experienced 4% less occurrence of crashes to their frequency (2% versus 6%). These results are possibly due to turning vehicles not having to give-way to pedestrians (there is not usually any kind of pedestrian facility at this treatment due to low number of pedestrians); or to conflicting traffic.

Left turn slip lanes experienced slightly lower occurrence of crashes in comparison to their relative volume exposure (51% versus 52%).

Conventional lanes

- The largest proportion of crashes occurred at shared lanes, although they experience noticeably less traffic volume exposure (28% < 36%). However, it was not statistically significant;

- ❖ Exclusive lanes experienced lower proportions of crashes compared to the shared lane but the proportion of crashes appeared to be substantially higher than their traffic volume exposure, which is counterintuitive to the results of the overall crash analysis. However, this was not statistically significant. The overall crash analysis indicated that exclusive lanes crashes were considerably smaller than their frequency (12% versus 21%). This is possibly due to a combination of the following reasons:
    - The under-representation issue of conventional lanes crashes that was highlighted in Chapter 4;
    - The small number of crashes found in overall crash analysis; and
    - The small sample size (267 approaches) used in the detailed analysis compared to overall analysis (1818 approaches), and particularly the ratio of exclusive lanes compared to shared lanes.

This issue could be worth further investigation in another research, to include left turn crashes for the whole signals network (including the 1818 left turn approaches).

Left turn conventional lanes experienced a slightly higher occurrence of crashes in comparison to their relative volume exposure (48% versus 49%).

In short, the traffic volume exposure analysis indicates that the crash rate for slip lanes was marginally less than conventional lanes; however, it was more likely to be similar (3.17 versus 3.23). Hence, it is perhaps reasonable to draw a conclusion that both the left turn slip lane and left turn conventional lane have comparable safety performance overall, but with some notable differences according to the treatment type. This was statistically verified.

5.2.3 Summary of crash rate

The crash rate for each left turn treatment in both analysis methods - frequency exposure and volume exposure - is summarised in Table 5-11.

Table 5-11 Left turn crash rate by frequency and volume exposure

Left Turn Treatment Type		Left turn crash rate	
		Frequency Exposure	Volume Exposure
Slip Lane	Signalised	1.12	6.42
	Free Flow	0.21	1.19
	Giveway No Pedestrian Marking	0.47	2.70
	Zebra Crossing	0.61	3.18
	Raised Zebra Crossing	0.00	0.00
	Slip Lane Sub Total	**0.57**	**3.17**
Conventional Lane	Exclusive Lane	0.71	5.53
	Shared lane	0.39	2.46
	Conventional Lane Sub Total	**0.48**	**3.23**
	Overall	**0.52**	**3.20**

The key findings:

- In both methods, the crash rate of each left turn treatment was different suggesting the various left turn treatments may differ in safety performance;
- Crash rate for the left turn slip lanes was the largest in the signalised slip lanes followed by zebra crossing slip lanes, while the free flow slip lane was the smallest in both frequency and volume exposures;
- Crash rate for the conventional left turn lanes was the greatest in exclusive lanes in both frequency and volume exposures, which is contrary to expectations; and
- Give-way slip lanes and shared lanes appeared to have a relatively similar crash rate for both frequency and volume exposure methods.

In both methods, the overall difference in the crash rate between left turn slip lanes and left turn conventional lanes was negligible. Consequently, the left turn slip lanes and the conventional left turn lanes have comparable safety performance which was statistically proven.

5.2.4 Pedestrian crash analysis

Pedestrians are vulnerable road users; therefore, it was desirable to examine pedestrian crashes separately from other road user crashes. Consequently, pedestrian crashes were extracted from the crash data set (the 140 left turn crashes). The crashes presented in Table 5-12 included pedestrian crashes only for the selected samples.

Table 5-12 Pedestrians involved in left turn crashes and treatment frequency

Left Turn Treatment Type		Left turn treatments frequency for selected samples		Left turn pedestrian crashes	
		No. of left turn treatments	% of left turn treatments	No.	%
Slip Lane	Signalised	17	6%	0	0%
	Free Flow	14	5%	0	0%
	Giveway No Pedestrian Marking	57	21%	0	0%
	Zebra Crossing	38	14%	2	33%
	Raised Zebra Crossing	0	0.0%	0	0%
	Slip Lane Sub Total	**126**	**47%**	**2**	**33%**
Conventional Lane	Exclusive Lane	41	15%	0	0%
	Shared lane	100	37%	4	67%
	Conventional Lane Sub Total	**141**	**53%**	**4**	**67%**
	Total	**267**	**100%**	**6**	100%

There was a total of 6 (4%) pedestrian crashes out of 140 that involved left turning movements. Two of these crashes occurred at zebra crossing slip lanes while the remaining four crashes occurred at shared conventional lanes.

Regardless of the small sample size, the results were analysed due to the importance of pedestrian crashes. The results indicate that the proportion of pedestrian crashes is considerably lower at slip lanes than their treatments frequency in the selected samples (33% versus 46%). Conversely, the largest proportion of pedestrian crashes occurred at shared conventional lanes compared to their frequency in the network (67% versus 37%).

This result is comparable to the overall pedestrian crash analysis results; nevertheless, the sample size of approaches was too small in this analysis, and, in contrast, the number of pedestrian crashes in one year was inadequate in the overall crash analysis task.

The pedestrian crashes were compared to their corresponding pedestrian crossing activations, to determine if the crashes occurring at these locations were due to a high level of pedestrian usage or not. Noted that where two pedestrian crashes occurred at zebra crossings, the nearby pedestrian crossing activations were chosen for the evaluation.

Table 5-13 indicates the comparison of pedestrian crashes with the number of pedestrian activations and level of usage. It clearly shows that 3 out 4 shared lanes pedestrian crashes occurred at relatively busy pedestrian crossings while only one occurred at crossings that have a low level of pedestrian usage. One of the two pedestrian crashes that occurred at a zebra crossing has a medium pedestrian usage level while the second crash has low pedestrian usage.

Table 5-13 Left turn pedestrian crashes corresponding to pedestrian activations

Intersection No.	CRASH ID	LT Crash Location	Ped No.	No. Ped Activation	Level of Usage
4137	201440960	SCL	P4	65	Low
2094	201101728	ZSL	P1	111	Low
2201	201335889	ZSL	P2	220	Medium
2109	201203515	SCL	P3	225	Medium
2094	201438433	SCL	P4	245	High
2378	201414936	SCL	P4	277	High

To overcome this sample size issue with pedestrian crashes, it was considered that an additional pedestrian crash analysis was needed. This means that pedestrian crashes should be collected and investigated for all signalised intersections in the network. This analysis will be carried out in the next chapter.

5.3 Summary

This chapter documents the data collection, procedure and analysis performed, to carry out the detailed crash analysis task. A list of 84 signalised intersections has been selected from the main intersection dataset that was produced in Chapter 3. These signalised intersections included a total of 267 approaches which allows left turns.

The 267 left turn approaches were classified according to the different left turn treatment types. Out of these approaches, there were 141 left turn conventional lanes and 126 left turn slip lanes.

The left turn crashes, collected for the five year period (2010-2014), were identified, and assigned to their respective left turn treatments.

Traffic volume for various left turn approaches were collected from SCATS detector loops. Furthermore, pedestrian activations from SCATS were collected for the approaches where pedestrian crashes occurred.

Pedestrian activation data was used to determine the level of exposure for pedestrian crashes and the traffic volume was used to calculate crash rate.

A number of analyses were carried out to determine the safety performance for various left turn treatments, particularly left turn slip lanes versus conventional left turn lanes.

The main crash analysis showed that a total of 144 crashes involved left turning movements. The key findings of the analysis are summarised in Table 5-14.

Table 5-14 Summary of left turn crashes for the selected 267 approaches, by frequency and volume exposures

Frequency Exposure		Left Turn Treatments	Volume Exposure	
% crashes vs. % frequency	Crash Rate		% crashes vs. % volume	Crash Rate
		Slip Lanes		
51% vs. 47%	**0.57**	**Overall**	**51% vs. 52%**	**3.17**
19% vs. 21%	0.47	give-way	19% vs. 23%	2.70
16% vs. 14%	0.61	zebra crossing	16% vs. 17%	3.18
14% vs. 6%	1.12	signalised	14% vs. 7%	6.42
2% vs. 5%	0.21	free flow	2% vs. 6%	1.19
		Conventional Lanes		
49% vs. 53%	**0.48**	**Overall**	**49% vs. 48%**	**3.23**
28% vs. 37%	0.39	shared	28% vs. 36%	2.46
21% vs. 15 %	0.71	exclusive	21% vs. 12%	5.53

The main findings were as follows:

- In the frequency exposure method, the left turn slip lanes experienced slightly higher occurrence of crashes in comparison to their relative treatments frequency, while left turn conventional lanes experienced a slightly lower occurrence of crashes in comparison to their relative treatments frequency;
- In the volume exposure method, the left turn slip lanes experienced slightly lower occurrence of crashes in comparison to their relative volume exposure. On the other hand, left turn conventional lanes experienced a slightly higher occurrence of crashes in comparison to their relative volumes;
- In the frequency exposure method, the crash rate for the left turn conventional lanes and the left turn slip lanes both have comparable crash rates 0.48 and 0.57 respectively. Consequently, the safety performance of both is almost similar; and
- In the volume exposure method, the crash rate for the left turn conventional lanes and the left turn slip lanes both have similar crash rates: 3.17 and 3.23 respectively. Thus, it is reasonable to draw a conclusion that the left turn slip lanes and the left turn conventional lanes have a comparable safety performance.

In summary, the proportion of crashes for left turn slip lanes and left turn conventional lanes was slightly different, but relatively similar in comparison to their exposures. Similarly, the crash rate of left turn slip lanes against left turn conventional was almost the same. Therefore, the left turn slip lanes and the conventional left turn lanes have similar safety, which was statistically proven in this research. Additionally, in the frequency exposure and volume exposure methods the crash rate was different at each of the left turn treatments. Hence, this suggests that the different types of left turn treatments may differ in their safety performance.

Noted that there were data and resources limitations in this research, thus other factors that may influence left turn slip lane performance were not investigated. For example the effect of the design features of different slip lanes treatments; whether it is a high entry angle or a low entry angle. Other factors include: approach gradient, pedestrian and traffic demands, and intervisibility. These factors should be explored in details in future research.

6 Additional Detailed Pedestrian Crash Analysis

The small sample size of left turn crashes involving pedestrians found in the previous analysis made it unfair to establish a conclusive answer to pedestrian safety at either left turn slip lanes or left turn conventional lanes.

Moreover, among most transport engineers there is a general disagreement around pedestrian safety, particularly at left turn slip lanes. Therefore, it was decided to carry out additional analysis and extend the research study further, to include pedestrian crashes for the entire signals network.

6.1 Pedestrian crashes for the whole signals network

This analysis was carried out on the 625 signalised intersections, 1818 approaches, which allow left turns. The analysis covered the five year period from 2010 to 2014 inclusive.

The main crash dataset obtained from CAS was filtered by pedestrian crashes using the same technique presented in Chapter 3.

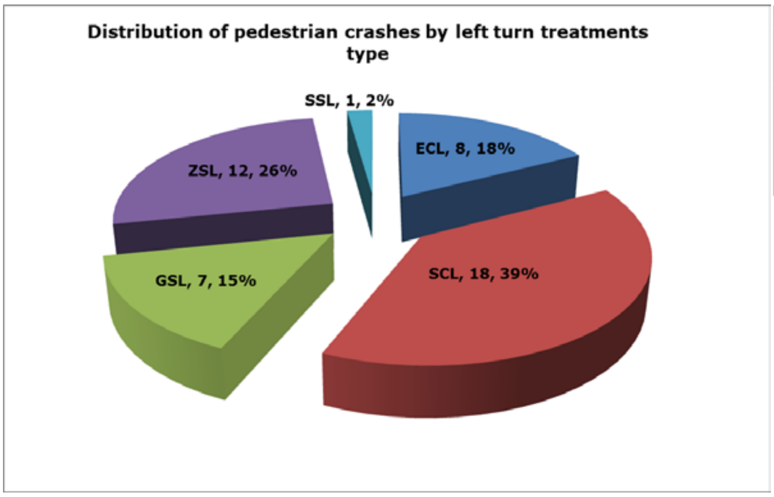
The TCR reports of these crashes encompassed approximately 800 pages. Accordingly, an extensive manual investigation for every pedestrian crash record was conducted, to determine whether the crashes involved left turning movements or not. Then these crashes were allocated to their relevant left turn treatments type.

The data of pedestrian crashes resulting from the initial manual examinations is indicated in Table 6-1. A total of 46 out of 209 pedestrian crashes were listed for further examination.

Table 6-1 Initial pedestrian crashes investigation

Preliminary pedestrian crash analysis	No of crashes	% of crashes
Left turn crashes involving pedestrians	46	22%
Crashes not related to left turn movements	163	78%
Total	**209**	**100%**

A result of the final detailed investigation of the 46 pedestrian crashes is shown in Figure 6-1. The figure indicates the distribution of pedestrian crashes resulting from left turn movements grouped by each left turn treatment type.



Code	Left Turn Type
ECL	Exclusive Conventional Lane
SCL	Shared Conventional Lane
FSL	Free Slip Lane
GSL	Giveway Slip Lane
ZSL	Zebra Slip Lane
SSL	Signalised Slip Lane

Figure 6-1 Distribution of pedestrian crashes by left turn treatments type for the whole signals network

6.2 Analysis method

Typically, the crash rate based on volume exposure should be established for this pedestrian analysis. However due to the large sample size involved, limited time and lack of resources, the frequency exposure method was used instead. This method was adapted in the O'Brien et al. (2010) study due to similar reasons.

6.3 Results of pedestrian crash analysis

Table 6-2 summarises the pedestrian crashes by various types of left turn slip lanes and left turn conventional lanes for the whole signals network (625 sites, 1818 approaches).

Table 6-2 Left turn pedestrian crashes, by type of treatments frequency in the entire signals network

Left Turn Treatment Type		Left turn treatments frequency in the signals network		Left turn pedestrian crashes by treatment type	
		No. of left turn treatments	% of left turn treatments	No.	%
Slip Lane	Signalised	75	4%	1	2%
	Free Flow	42	2%	0	0%
	Giveway No Pedestrian Marking	460	25%	7	15%
	Zebra Crossing	163	9%	12	26%
	Raised Zebra Crossing	1	0%	0	0%
	Slip Lane Sub Total	**741**	41%	**20**	**43%**
Conventional Lane	Exclusive Lane	389	21%	8	17%
	Shared lane	688	38%	18	39%
	Conventional Lane Sub Total	**1077**	59%	**26**	**57%**
	Total	**1818**	100%	**46**	100%

Key findings:

- ❖ The overall proportion of left turn pedestrian crashes is very small compared with the relevant treatments frequency in the signals network (46 versus 1818). This resulted in a crash rate of only 2.35% in a five year period. Possibly this is due to the low speeds of the vehicles decelerating to turn left.
- ❖ The crash rate for the slip lanes (0.027; 2.7%) was slightly higher than for conventional lanes (0.024; 2.4%). However, the difference between the two crash rates is considered to be minimal, especially for a five year period. Therefore, the overall safety performance for both slip lanes and conventional could be considered similar; this was statistically proven;
- ❖ Shared conventional left turn lanes pedestrian crashes formed the largest proportion of crashes, followed by zebra crossing slip lanes, give-way slip lanes, and then exclusive conventional lanes. In contrast, the smallest proportion of left turn pedestrian crashes occurred at signalised slip lanes, free flow and raised zebra crossing slip lanes with no crash records;
- ❖ The signalised slip lanes experienced 2% lower pedestrian crashes compared to their share of treatment frequency. This facility is usually provided at locations that have low pedestrian usage and high left turn volumes (mostly two signalised lanes, and major intersections);
- ❖ Shared left turn lanes experienced a marginally higher proportion of pedestrian crashes compared to their share of treatment frequency. However, this was not statistically significant;

  Exclusive left turn lanes experienced a lower proportion of pedestrian crashes compared to their treatment frequency (17% versus 21%). This treatment implementation requires a large land area, thus it is usually used in lower pedestrian areas; consequently pedestrian exposure is slightly low;

- Free flow slip lanes appear to be considerably safer with no pedestrian crashes. Nevertheless, this could be due to the low pedestrian usage;
- Zebra crossing slip lanes experienced a 17% higher occurrence of pedestrian crashes compared to their share of treatment frequency in the network. This was statistically proven to be significant. Therefore, it is reasonable to conclude that zebra crossings have the worst safety performance compared to other left turn treatments type. These results could be influenced by other factors. For example, zebra crossings are usually implemented in areas with high pedestrian demands; therefore, the pedestrian exposure is high. However, the use of zebra crossing with a raised platform might increase its safety performance which was not the case in this study; and
- The give-way slip lanes accounted for 10% lower occurrence of pedestrian crashes compared to their frequency within the network. Hence, the give-way treatment is considerably safer than zebra crossings when it comes to selecting which treatment option should be chosen for left turn slip lanes. This could be due to the fact that give-way slip lanes are usually used in low pedestrian demand areas, where pedestrian exposure is low. On the contrary, the difference in the results was not proven to be statistically significant.

In summary, the left turn slip lanes may have similar safety performance to the left turn conventional lanes without taking into account pedestrian volume exposure and the key design features at each treatment. Additionally, the results highlighted that the zebra crossing slip lanes performed poorly in terms of pedestrian safety, which was statistically proven.

Interesting further research can be conducted on slip lanes design features with topics to additionally subdivide the left turn slip lanes into more categories including:

- Left turn slip lane types: high entry angle or low entry angle;
- Island size and shape;
- Deceleration lane length; and
- Positions of pedestrian crossings at slip lanes.

6.4 Pedestrian crashes by severity

Figure 6-2 depicts the severity of left turn pedestrian crashes for various left turn treatments. It also shows the combined injury crashes (minor, serious, and fatal) versus non-injury crashes.

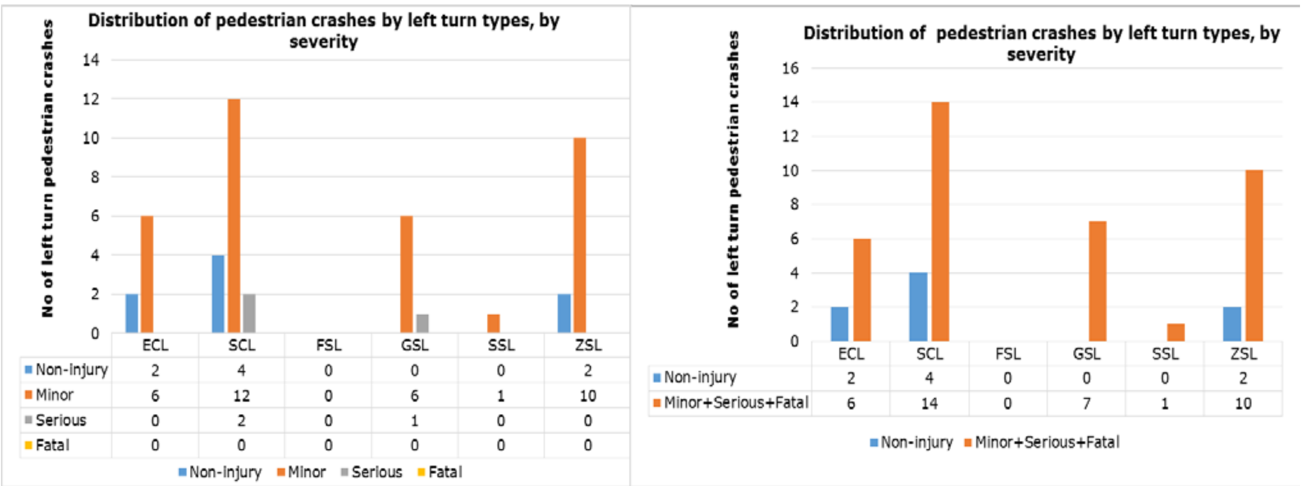


Figure 6-2 Distribution of left turn pedestrian crashes, by severity and treatments type

Main findings looking at both charts:

- The greatest proportion of left turn pedestrian crashes resulted in minor injuries, followed by non-injuries. This might be due to fact that the left turn pedestrian crashes tend to involve moderately low speeds when vehicles are decelerating to turn left;
- The smallest proportion of left turn pedestrian crashes were serious injuries while no fatal crashes were recorded;
- Slip lanes and conventional lanes experienced almost the same proportion of pedestrian injury crashes;
- The largest proportion of pedestrian injury crashes occurred at conventional left turn lanes, predominantly at the shared lanes; and
- The zebra crossing experienced the highest proportion of pedestrian injury crashes for left turn slip lane treatment.

6.5 Pedestrian crashes by movement codes

The distribution (numbers and percentages) of left turn pedestrian crashes at conventional lanes versus slip lanes is shown in Figure 6-3. The crashes were not disaggregated by each type of left turn treatment due to the small sample size of crashes, so they were grouped into two categories: slip lanes and conventional lanes. This graph depicts the four individual crash movement codes: NA, NB, NC and NE. Also, it depicts two groups of crashes: left side (NA+NC) and right side (NB+NE); this is to identify any specific patterns that emerge. The description of each crash movement code can be found in Appendix A.

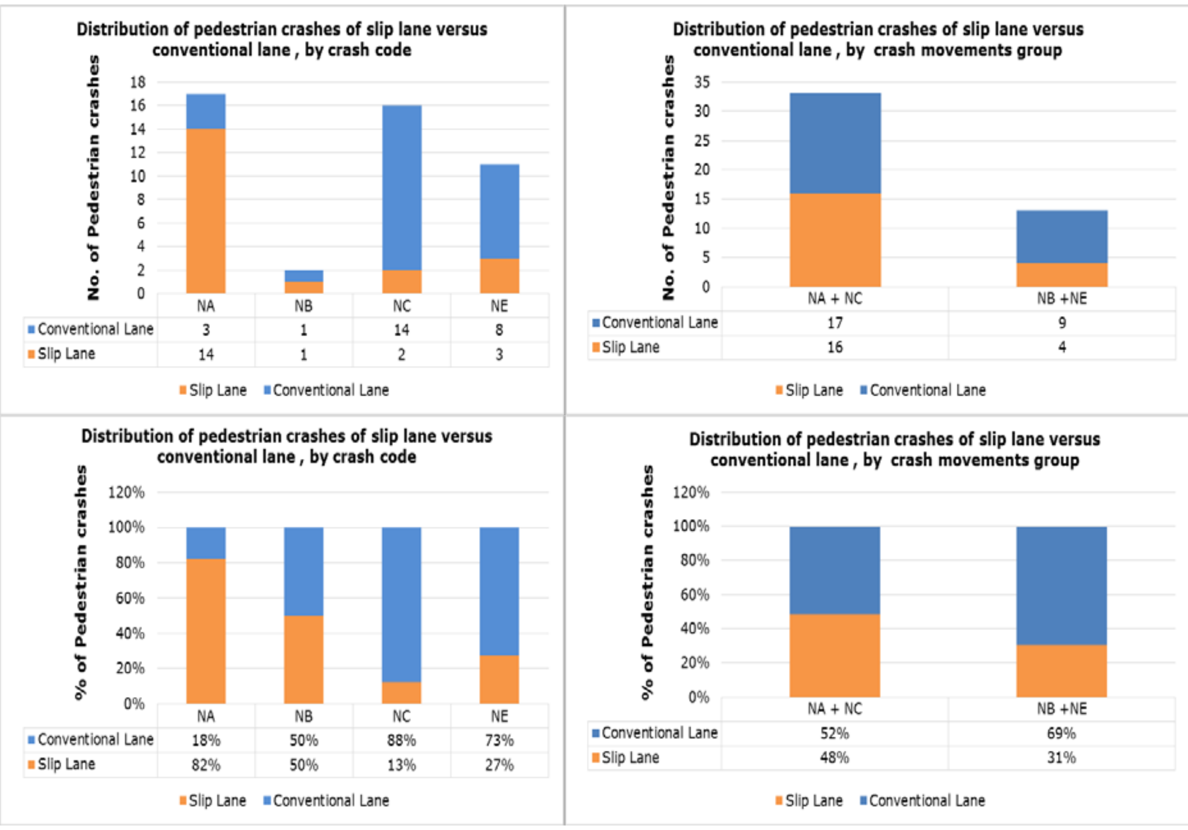


Figure 6-3 Distribution of pedestrian crashes by movement codes at conventional lanes versus slip lanes

Key finding:

- Pedestrian crashes that occurred at conventional lanes were dominated by NC type, followed by NE type.
- Pedestrian crashes recorded at the slip lanes were prevalent by NA type, followed by NE type.
- The majority of left turn pedestrian crashes involved pedestrian crossings from the left side (NA+NC group) when hit by left turning vehicles. Additionally, this pedestrian crash movement group has a similar proportion of crashes at both slip lanes and conventional lanes;

- ❖ The minority of left turn pedestrian crashes involved pedestrians crossing from the right side (NB+NE group) when struck by vehicles turning left. This occurred at the conventional lanes rather than at the slip lanes; and
- ❖ The smallest proportion of pedestrian crashes occurring at both slip lanes and conventional lanes was the NB type.

6.5.1 Discussion on slip lanes pedestrian crashes

Pedestrian crashes were predominated by the NA pedestrian to vehicle conflict type. This conflict occurs when drivers approaching slip lanes are looking at their right-hand side to find an appropriate gap in the conflicting traffic movements, and failing to notice the pedestrian crossing the slip lane from their left-hand side. This is illustrated in Figure 6-4. This safety issue can be alleviated by improving the design of the slip lane.

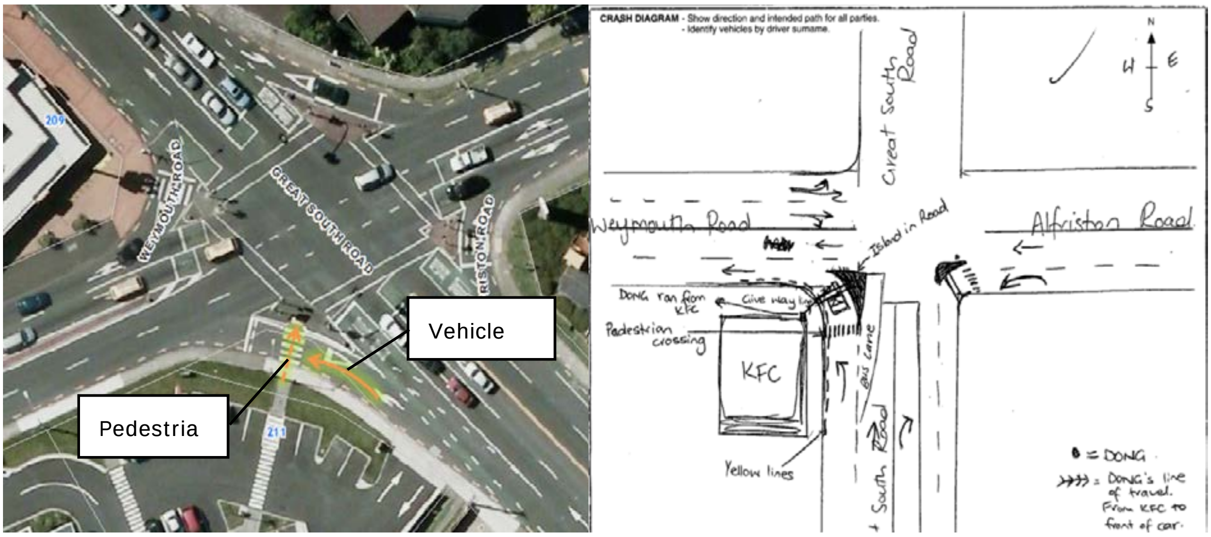


Figure 6-4 Example of pedestrian crash (crash ID: 201004225) illustrates NA crash type

6.5.2 Discussion on conventional lanes pedestrian crashes

Pedestrian crashes were dominated by the NC pedestrian to vehicle conflict type. These crashes occur when pedestrians are struck by vehicles turning left whether the pedestrian crossing signals are operating or not, and pedestrians are crossing from the left-hand side, as illustrated in Figure 6-5 . In this situation, it is basically either vehicles or pedestrians running a red light. Another reason is that the turning vehicles do not notice, or do not give-way to pedestrians.

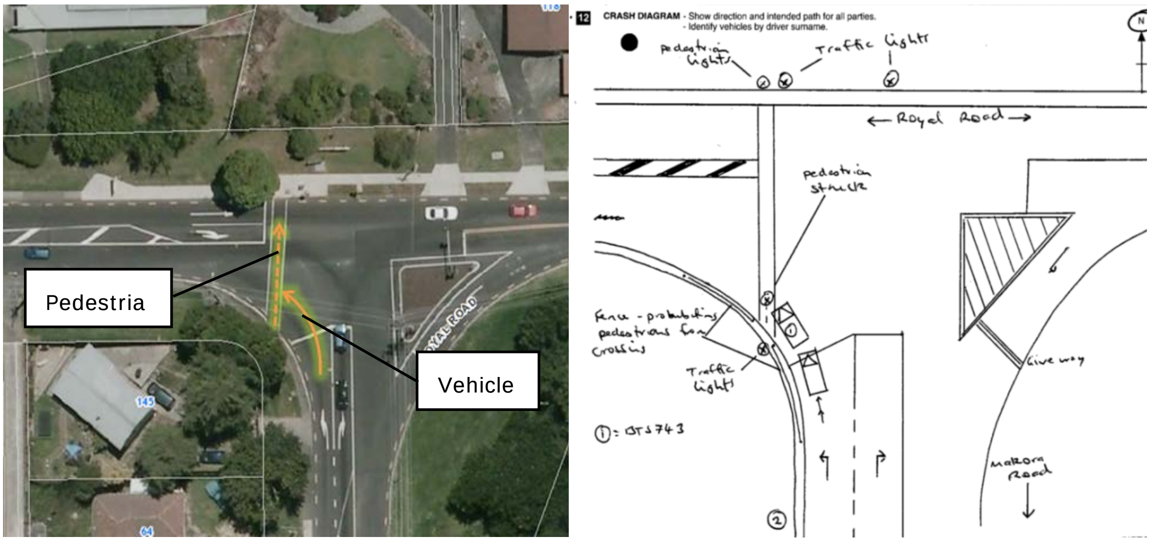


Figure 6-5 Example of pedestrian crash (crash ID: 201102747) illustrates NC crash type

Another piece of a research which can be conducted in this area is by collecting more samples of left turn approach and disaggregate pedestrian crashes by various left turn treatment types, and crash movement codes.

6.6 Comparison study

An interesting comparison deemed to be beneficial in this study was to compare the results that were presented in Table 6-2 (referred to as the Auckland study) with the O'Brien et al. (2010) study (referred to as the Melbourne study). It is worth noting that the Melbourne study included a very large sample size as well as implying different road rules, especially around give-way control rule at left turn slip lanes; however, it gives a reasonably comparable analysis. The results of the Melbourne study were re-arranged to enable a comparison between the two results. The result of this comparison analysis is shown in Table 6-3.

Table 6-3 Comparison study of pedestrian crashes in Auckland compared to Melbourne

Auckland Study No. of signalised sites: 625 No. of approaches: 1818 5 Years					Melbourne Study (VicRoads) O'Brien et al. (2010) No. of signalised sites: 2284 No. of approaches: 6987 5 Years				
Left Turn Treatment Type	Left turn frequency treatments		Left turn pedestrian crashes		**Left Turn Treatment Type**	Left turn frequency treatments		Left turn pedestrian crashes	
	No.	**%**	**No.**	**%**		**No.**	**%**	**No.**	**%**
Signalised	75	4%	1	2%	L Slip sig	292	4%	5	3%
Free Flow	42	2%	0	0%	L Slip-free slip	31	0%	0	0%
Giveway No Pedestrian Marking	460	25%	7	15%	L Slip Unsig	629	9%	11	6%
Zebra Crossing	163	9%	12	26%	L Slip zebra	1129	16%	26	13%
Raised Zebra Crossing	1	0%	0	0%	N/A	0	0%	0	0%
Slip Lane Sub Total	**741**	**41%**	**20**	**43%**	**Slip Lane Sub Total**	**2081**	**30%**	**42**	**22%**
Exclusive Lane	389	21%	8	17%	Exclusive Lane	1407	20%	36	18%
Shared lane	688	38%	18	39%	Shared lane	3499	50%	117	60%
Conventional Lane Sub Total	**1077**	59%	**26**	**57%**	**Conventional Lane Sub Total**	**4906**	70%	**153**	78%
Total	**1818**	**100%**	**46**	**100%**	**Total**	**6987**	**100%**	**195**	**100%**

Key findings:

- Generally, the proportion of left turn crashes compared to their total number of approaches in both the Melbourne and the Auckland studies were similar: 2.7% and 2.5% respectively. Hence, the proportion of left turn crashes involving pedestrians was generally very small.

- The two studies appear to have similar proportions of pedestrian crashes that occurred at signalised left turn slip lanes and exclusive left turn conventional lanes: 2% for the Auckland study and 3% for Melbourne study;

- The shared lanes appear to have considerably higher pedestrian crashes (about 10%) compared to their frequency treatment on the network in the Melbourne study. In contrast, shared lanes experienced the highest proportion of pedestrian crashes among other treatments in Auckland; in the Auckland study it was statistically insignificant;

- Both studies indicated that free flow (equivalent to L-slip free slip) slip lanes are comparatively safe with no recorded crashes;

- In the Melbourne study, both unmarked (L Slip Unsig) and zebra crossing slip lane treatments showed lower crashes (3%) when compared to their relevant treatment frequency. This is dissimilar to the Auckland study which indicated that these treatments experienced a substantial proportion of pedestrian crashes compared to their frequency treatment. In particular, zebra crossing slip lanes accounted for a larger proportion of crashes compared to their treatment frequency (26% versus 9%), which was statistically significant. The different results may be due to zebra crossing treatments at left turn slip lanes in Melbourne having several configuration, some of which are:

- o Zebra crossings with and without flashing light; and
- o Wombat type zebra crossing with or without flashing light.

In the Melbourne study these configurations were grouped under one category called zebra crossings. As a result, the two studies in this particular case were not comparable.

In short, the overall comparison analysis between the two studies indicates that there are some similarities in most of the results. The two studies have a similar proportion of left turn crashes compared to the total number of approaches. Both agree that the proportion of left turn crashes involving pedestrians is generally very small. However, the Melbourne study indicated that the left turn slip lanes are safer than the left turn conventional lanes for pedestrians; on the contrary, in this research both have similar safety performance.

6.7 Slip lanes versus conventional lanes design

6.7.1 Intersections designed with left turn conventional lanes

Left turn slip lanes when removed and replaced with conventional lanes results in pedestrians having to cross the full leg of an intersection in one crossing phase.

This makes the assumption that pedestrians are safer on a longer, fully controlled crossing, with left turning traffic allowed to filter through the parallel signalised pedestrian crossing, than crossing a single slip lane uncontrolled (Evans, 2008).

This may possibly occur at the expense of the following:

- ❖ Increasing the number of potential conflict points within the signalised intersection area;
- ❖ Increasing the overall conflict area within the signalised intersection. Usually for heavy vehicles, the left turn tracking paths required can result in a very large overall signalised intersection areas, and can result in very long pedestrian crossings. By contrast, slip lanes give a shortened pedestrian crossing length within the signalised part of the intersection, and reduce the intersection conflict area. As indicated in Figure 6-6, the intersection area increased from 316m^2 to 596m^2 when slip lanes were removed and replaced with conventional lanes;

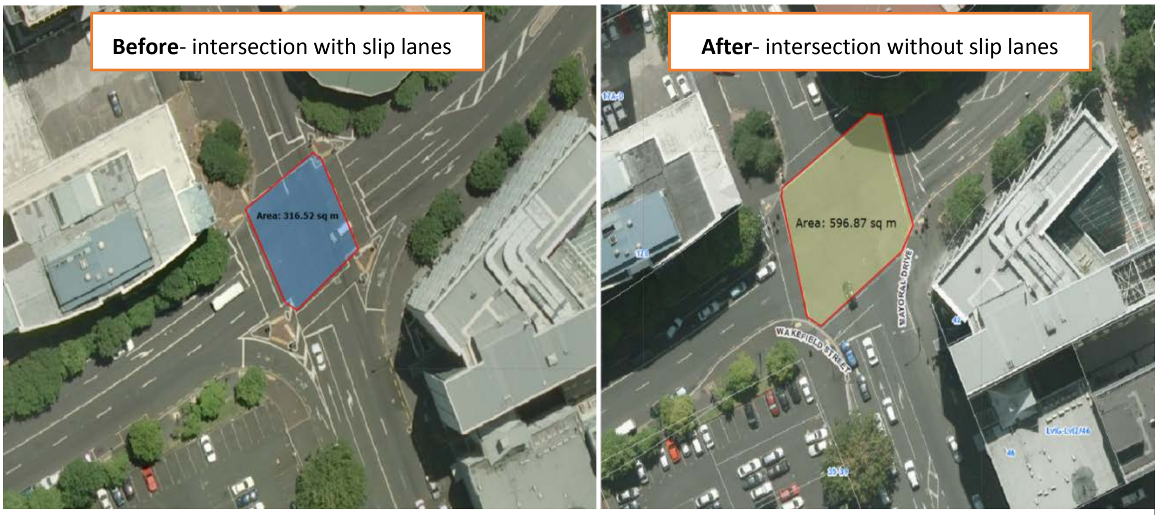


Figure 6-6 Intersection size before and after slip lanes removal

❖ Increased pedestrian crossing lengths expose pedestrians to filtering traffic for significant periods. These are dissimilar to slip lanes where pedestrians cross a short single lane and thus the exposure periods are very short approximately 3-6 seconds compared to 15-25+ seconds for conventional lanes, as indicated in Figure 6-7.

Figure 6-7 Pedestrian exposure time to left turning traffic at slip lanes versus conventional lanes

❖ Increased pedestrian crossing length, results in longer cycle times. Hence increasing the overall waiting time for the introduction of pedestrian phases and reducing the overall intersection capacity. Inefficient signal operation usually results in pedestrians either crossing against red signals or crossing outside the controlled area; both issues can lead to increasing risk of crashes;

- ❖ Increased intersection areas require longer vehicle clearance periods for through traffic; consequently, increases the potential for red light running conflicts at the end of a phase;
- ❖ Increased intersection areas usually lead to locating signal displays further away from the driver line of sight, hence reducing the overall effectiveness of the signal displays and increasing the crash risk; and
- ❖ Left turn vehicles within the signalised intersection area demand a green signal phase; as a result, this increases the number of phase changes occurring during periods of very low traffic demand. This perhaps increases conflict potential and driver frustration.

6.7.2 Intersections designed with left turn slip lanes

Intersections designed with left turn slip lanes require pedestrians to cross more than one carriageway while traversing a single leg of an intersection. Even though there are safety concerns about the need to cross uncontrolled left turn movements, there are a few best practice design features that are being used that could improve safety performance of left turn slip lanes. Some of these features are as follows:

Angel of Entry

- ❖ Generally left turning traffic enters the slip lane at slow speed. However, there are a number of parameters that contribute to this speed including:
  - o The angle between the approach road and the departure road;
  - o The angle of the slip lane; and
  - o Intervisibility between left turning drivers approaching the slip lanes and pedestrians at the crossing points.
- ❖ Crossing slip lanes is sometimes difficult for some pedestrians. This is due to the safety issue of drivers (human factor) concentrating on opposing cross road traffic (looking to the right) rather than focusing on opposing crossing pedestrians. However, this issue can be mitigated by improving the geometry of the slip lane design as shown in Figure 6-8.

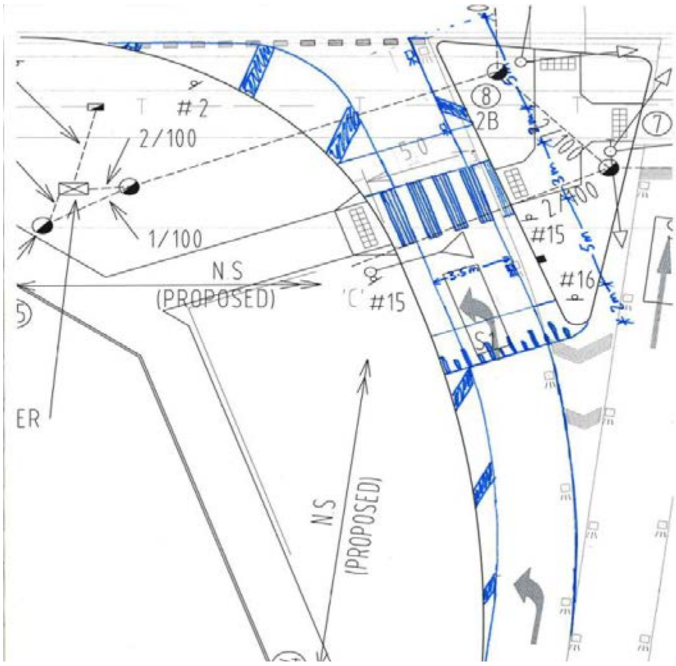


Figure 6-8 Example of the slip lane design improvement (O'Brien et al., 2010)

Therefore, it is recommended that the angle of entry between the slip lanes and the cross street be 70 to 90 degrees. This high entry angle promotes slower speeds of motorist. As well as it encourages drivers to focus on the pedestrian crossing, rather than turning their head to the right to look for gaps in the approaching traffic.

Splitter Island

- ❖ Island size could be an issue for pedestrian safety, if it is not large enough to accommodate pedestrians. This issue can be mitigated by increasing the island size where possible. Also, the island size should be adequate to draw the motorist's attention.

Deceleration Lane:

- ❖ Deceleration lanes are usually expected to slow down left turning vehicles, depending on lane width and kerb radius. This may lead to the reduction of vehicle crashes especially rear-end crashes by separating left turning traffic from through traffic. Moreover, this separation could help pedestrians to identify left turn vehicles.

Crosswalk Location:

- ❖ The crosswalk should be located at the upstream end of the splitter island and set back at least 6 meters from the give way line. This is to maximise intervisibility between pedestrians and approaching drivers, as well as making the crossing prominent. In addition it provides storage for one vehicle in downstream of the crossing.

Raised Platform:

- Place the zebra crossing on a raised platform, forcing drivers to slow down, improves the visibility of the crossing and make it more conspicuous.

The safety concerns or safety perception of slip lanes can be mitigated by many tools. These tools are explored fully in the O'Brien et al. (2010) study, Chapter 13 ”*Tools For Improving Left Turn Facilities”,* which particularly provides a wide range of options on how to improve safety of pedestrians at left turn slip lanes as well as conventional lanes. The present author recommends this chapter is adopted into this study for implementation and testing in the New Zealand environment. In doing so, the author suggests that each case is assessed on its own merits rather than applying a blanket policy of not having left turn slip lanes.

The following key benefits of left turn slip lanes are summarised as follows (Evans 2008):

- Reduces number of conflict points within the signalised intersection area;
- Reduces signalised intersection conflict area;
- Shortens pedestrian crossing length and providing fully protected pedestrian crossings within the signalised area. It is worth noting that it results in shorter signal cycle time and will reduce delays for all users at the intersections, enabling optimal positioning of signal displays, and hence improve safety;
- Introduces more phasing opportunities for pedestrians such as walk for green, which allows the re-introduction of a pedestrian crossing even after the parallel traffic phase has started, as long as there is adequate time in the running phase. This reduces the delay for pedestrians waiting to cross and hence reduces the risk that pedestrians may be crossing against the red signal; and
- Reduces the overall intersection delay, especially the left turn movements.

In summary, slip lanes have many safety and efficiency advantages in comparison to conventional lanes as they provides an overall reduction in intersection conflict, enable optimal location of signal displays, reduce delay to pedestrians and reduce driver frustration especially during off peak times. Consequently, they improved safety and operational performance of the intersection, and reduced crash risk is realised.

6.8 Summary

This analysis included 625 signalised sites comprising 1818 approaches that enable left turns. This analysis covered a five year period (2010 to 2014 inclusive). There was a total of 46 left turn vehicle versus pedestrian crashes at signalised sites. Pedestrian crashes occurring in relation to different left turn treatments in comparison to their treatment frequency are summarised as follows:

Slip Lanes

- 26% versus 9% for zebra crossing slip lanes;
- 15% versus 25% for give-way slip lanes; and
- 2% versus 4% for signalised slip lanes.

Conventional Lanes

- 39 % versus 38% for shared conventional lanes; and
- 17% versus 21 % for exclusive conventional lanes.

In summary, the left turn slip lanes have similar safety performance to the left turn conventional lanes, without taking into account pedestrian volume exposure and the key design features at each treatment. Additionally, the results highlighted that the zebra crossing slip lanes performed poorly in terms of pedestrian safety. These results were statistically confirmed.

It is important to note that these results could be influenced by other factors that were not included in this research, due to data and resources limitations. One of these factors is the geometry design of slip lanes, which can result in some treatments being safer than others. Another factor is the pedestrian and traffic demands, which usually governs the use of different slip lane treatments type. Hence, the pedestrian exposure can increase at certain treatments rather than others. These features can be explored further in another research.

Further analysis on left turn pedestrian crashes was carried out by *severity* and by *crash movement codes*. Key findings are summarised as follows:

- The greatest proportion of left turn pedestrian crashes resulted in minor injuries, followed by non-injuries, and a small proportion were serious injuries. Note that there were no fatal crashes recorded. This is expected because left turn pedestrian crashes tend to involve moderately low speeds when vehicles decelerate to turn left;
- Slip lanes experienced fewer injury crashes than conventional lanes;
- The largest proportion of pedestrian injury crashes predominantly occurred at shared lanes for conventional lanes and at zebra crossings for slip lanes;
- The pedestrian crashes occurring at conventional lanes were dominated by NC type, followed by NE type; and

- The pedestrian crashes occurred at slip lanes which were overwhelmingly predominated by NA type.

There are a few design features that are being used that could improve the safety performance of left turn slip lanes. Some of these features are: the use of high entry angle, adequate island size and placing zebra crossing on a raised platform. However the effect of implementing these design features on the safety performance should be explored in a further research.

Comparison study

A comparison study between the O'Brien et al. (2010) study (referred to as the Melbourne study) and the results of the left turn pedestrian crashes (the Auckland study) was conducted and results analysed.

The overall comparison analysis between the two studies indicates that there are some similarities in most of the results. The two studies have a similar proportion of left turn crashes compared to the total number of approaches. Also, both agree that the proportion of left turn crashes involving pedestrians is generally very small. Additionally, the two studies appear to have similar proportions of pedestrian crashes occurring at signalised left turn slip lanes and exclusive left turn conventional lanes. Moreover, free flow slip lanes in both studies were considered safe as no crashes were recorded.

On the contrary, the proportion of pedestrian crashes occurring at zebra crossings was substantially different in both studies. In the Melbourne study the proportion of crashes was lower than that of the Auckland study. The shared lanes in the two studies experienced higher pedestrian crashes than their frequency in the network;

In summary, the Melbourne study showed that the left turn slip lanes are safer than the left turn conventional lanes for pedestrians, which is contrary to this research, where both have similar safety performance; this was statistically confirmed.

7 Statistical Analysis

This chapter presents the statistical analysis that was undertaken on the crash analysis results presented previously. The Chi-square test was chosen to measure the statistical significance of the results, due to the categorical nature of the data used and sample size of this research.

7.1 Chi-Square Test

Generally, there are several types of Chi-square test that can be used, depending on the way the data was collected and the hypothesis being tested. In this research, the two-dimensional arrangement was used as a contingency table 2x2 shown in Table 7-1. The test designed to examine the relationship between rows variable and columns variable. It can be either a test for equality or independence of two variables.

Table 7-1 General notation for a 2X2 contingency table

	Data Type1	Data Type2	Row Totals
Category 1	A	C	**m**=a+c
Category2	B	D	**g**=b+d
Column Totals	**r**=a+b	**s**=c+d	**N**=a+b c+d

In this table the letters a, b, c and d denote the contents of the cells. These letters are used to predict what count would be expected for each cell if the null hypothesis were true.

The expression used for chi-square test (X^2) is shown as below;

$$x^2 = \frac{N(ad-bc)^2}{mgrs}$$

A Chi-square test value (X^2) of 6.63 indicates that the null hypothesis is true with 99% confidence level, while a test value of 3.38 indicates a 95% confidence level, and a test value of 2.70 indicates a 90% confidence level. The degree of freedom (df) for a 2x2 contingency table is 1 for all confidence levels. The three confidence levels, 99%, 95%, and 90% were used in the analysis.

7.2 Chi-Square statistical results

The Chi-square test was used to measure the statistical significance of the crash analysis results obtained in the research. It was used on the detailed crash analysis and additional detailed pedestrian crash analysis. This was to confirm or deny that there is a statistically significant difference in the proportion of observed and expected left turn crashes between any two groups of the left turn treatments.

To conduct the Chi-square test, the following two sets of data were required:

- The number of crashes observed for the particular type of any of the left turn treatments; and
- The number of crashes expected for the particular type of any of the left turn treatments. This number was calculated using the following equation;

$$Expected\ number\ of\ crashes\ = \frac{Total\ No.of\ observed\ crashes\ \times No.of\ LT\ approaches}{Total\ No.of\ LT\ approaches}$$

The Chi-square test is used to test for the equality of the relative proportions of observed and expected crashes between two groups of the left turn treatments type.

The hypotheses used are:

- Null Hypothesis H0: the relative proportions of the observed and the expected left turn crashes are the same; or
- Alterative Hypothesis H_A: the relative proportions of the observed and the expected left turn crashes are NOT the same.

In the following sections, the main two groups for each analysis, slip lanes and conventional lanes, were compared against each other. A comparison was also conducted on the different other treatments against their counterpart of the same category for the slip lane treatments. Moreover, any combinations that had a notably higher or lower proportion of crashes than their treatment frequency were compared against all other treatments.

7.2.1 Detailed crash analysis statistical results

The Chi-square test was carried on the detailed crash analysis results presented in Table 5-9, Chapter 5. The outcomes of results are indicated in Table 7-3.

Table 7-2 Chi-square results for the detailed crash analysis

	Data used				Results				
	No.of LT Approaches	No. of **Observed** Crashes	No. of **Expected** Crashes	Row Totals	x^2	p-value	Is it significant at **99%** (Y/N)	Is it significant at **95%** (Y/N)	Is it significant at **90%** (Y/N)
LT slip lanes versus LT conventional lanes					0.5144	0.473244	N	N	N
Slip lanes	126	72	66	138					
Conventional lanes	141	68	74	142					
Column Totals	267	140	140	280					
LT signalised slip lanes versus other slip lanes treatments type					**3.4975**	**0.06146**	N	N	Y
Signalised slip lanes	17	19	10	29					
Other slip lanes treatments type	109	53	62	115					
Column Totals	126	72	72	144					
LT give-way slip lanes versus other slip lanes treatments type					1.0286	0.310494	N	N	N
Give-way slip lanes	57	27	33	60					
Other slip lanes treatments type	69	45	39	84					
Column Totals	126	72	72	144					
LT zebra crossing slip lanes versus all other slip lanes type					0.0323	0.85732	N	N	N
Zebra slip lanes	38	23	22	45					
Other slip lanes treatments type	88	49	50	99					
Column Totals	126	72	72	144					
LT free flow slip lanes versus other slip lanes treatments type					2.4607	0.116727	N	N	N
Free flow slip lanes	14	3	8	11					
Other slip lanes treatments type	112	69	64	133					
Column Totals	126	72	72	144					
LT signalised slip lanes versus All other left lanes treatments type					**3.9683**	**0.04637**	**N**	Y	Y
Signalised slip lane	17	19	9	28					
All treatments type	250	121	131	252					
Column Totals	267	140	140	280					
LT give-way slip lanes versus All other left lanes treatments type					0.1983	0.656134	N	N	N
Give-way slip lanes	57	27	30	57					
All treatments type	210	113	110	223					
Column Totals	267	140	140	280					
LT zebra slip lanes versus All other left lanes treatments type					0.2473.	0.618999	N	N	N
Zebra slip lanes	38	23	20	43					
All treatments type	229	117	120	237					
Column Totals	267	140	140	280					
LT shared conventional lanes versus LT exclusive conventional lanes					2.5841	0.107942	N	N	N
Shared lanes	100	39	48	87					
Exclusive lanes	41	29	20	49					
Column Totals	141	68	68	136					

Results of the key findings:

- Left turn slip lane showed **no** statistical significance at the three confidence levels tested when compared to left turn conventional lanes. Hence the two groups have the same proportion of crashes;

- Give-way slip lanes, zebra crossing slip lanes and free flow slip lanes revealed **no** statistical significance at the three confidence levels tested when compared with other slip lane treatments. Accepting the null hypothesis, and that the relative proportions of the observed and the expected left turn crashes are the same between the two groups;
- Give-way slip lanes and zebra crossing slip lanes showed **no** statistical significance at the three confidence levels tested when compared with **all** other left turn treatments type. Accepting that give-way and zebra crossing have the same proportion of left turn crashes when compared with all other left turn treatments;
- Shared conventional lanes versus exclusive conventional lanes proven to be **no** statistical significance at the three confidence levels tested. Therefore, the shared and the exclusive lanes have the same proportion of left turn crashes; and
- Signalised slip lane versus other slip lane treatments type found it was **statistically significant** at 90% confidence level. It was also statistically significant at 95% when compared with **all** other left turn treatments. Therefore, the signalised slip lanes performed poorly in terms of the proportion of crashes when compared with other slip lane treatments as well as **all** other left turn treatments.

7.2.2 Pedestrian crash analysis statistical results

The Chi-square test was carried out on the pedestrian crash analysis results presented in Table 6-2, Chapter 6. The outcomes of results are in Table 7-4.

Table 7-3 Chi-square results for the additional pedestrian crash analysis

Data used					Results				
	No.of LT Approaches	No. of **Observed** Crashes	No. of **Expected** Crashes	Row Totals	x^2	p-value	Is it signficant at **99%** (Y/N)	Is it signficant at **95%** (Y/N)	Is it signficant at **90%** (Y/N)
LT slip lanes versus LT conventional lanes					0.0445	0.83291	N	N	N
Slip lanes	741	20	19	39					
Conventional lanes	1077	26	27	53					
Column Totals	1818	46	46	92					
LT shared conventional lanes versus exclusive conventional lanes					0.0461	0.829966	N	N	**N**
Shared lanes	688	18	17	35					
Exclusive lanes	389	8	9	17					
Column Totals	1077	26	26	52					
LT give-way slip lanes versus zebra crossing slip lanes					2.5063	0.113394	N	N	N
Give-way slip lanes	460	7	14	21					
Zebra crossing slip lanee	163	12	5	17					
Column Totals	623	19	19	38					
LT zebra crossing slip lanes versus other slip lanes treatments type					**6.6667**	**0.00982**	**Y**	**Y**	**Y**
Zebra slip lanes	163	12	4	16					
Other slip lanes treatments type	578	8	16	24					
Column Totals	741	20	20	40					
LT zebra crossing slip lanes versus conventional lanes treatments type					**6.6667**	**0.00982**	**Y**	**Y**	**Y**
Zebra slip lanes	163	12	5	17					
Conventional lanes	1077	26	33	59					
Column Totals	1240	38	38	76					
LT zebra slip lanes versus All other left lanes treatments type					**4.8421**	**0.02777**	**N**	**Y**	**Y**
Zebra slip lanes	163	12	4	16					
All treatments type treatments	1655	34	42	76					
Column Totals	1818	46	46	92					

Results of the key findings:

- There was no statistical significance demonstrated when comparing slip lanes against conventional lanes at the three confidence levels tested. Thus, accepting the null hypothesis, and the relative proportions of the observed and the expected left turn crashes are the same for the two groups;
- Shared conventional lanes demonstrated **no statistical significance** at the three confidence levels tested when compared with exclusive conventional lanes. Therefore, the proportion of crashes is the same for the two groups;
- Give-way slip lanes demonstrated **no statistical significance** at the three confidence levels tested when compared with zebra crossing slip lanes. Thus, the proportion of crashes is the same for the two groups; and

- Zebra crossing slip lanes were found to be **statistically significant** at the three confidence levels tested when compared to other left turn slip lanes and conventional lanes. Furthermore, it was found to be statistically significant at 95% and 90% confidence level when compared with **all** other left turn treatments. Rejecting the null hypothesis, and accepting that the relative proportions of the observed and the expected left turn crashes are NOT the same between the zebra crossing and the other groups. Consequently, the zebra crossing slip lanes are the worst left turn treatments at signalised intersections for pedestrians.

8 Intersection Operational Performance

This chapter presents the methodology and results of the operational performance analysis of various left turn treatments.

8.1 Traffic operational modelling

SIDRA INTERSECTION 6.1 (referred to as SIDRA) modelling software was chosen to undertake the traffic modelling assessment for intersection performance. SIDRA is an industry standard software package used worldwide for planning professionals. It is an advanced micro-analytical tool for evaluation of intersection performance in terms of capacity, delay, degree saturation, level of service and queue length for vehicles and pedestrians.

The aim of the SIDRA traffic modelling analysis was to assess the traffic operational performance of four different left turn treatments at a signalised intersection. These are:

- Slip lane with give-way control, with no pedestrian crossing marking;
- Slip lane signal control, with signalised pedestrian crossing;
- Conventional exclusive lane; and
- Conventional shared left lane.

These left turn treatments type were chosen because they represent the most typical signalised intersections, in which a slip lane with give-way control is generally utilised in one of the following ways:

- A slip lane with a give-way control; or
- The slip lane is being removed and replaced with a signalised slip lane, an exclusive lane or a shared lane.

8.2 Site location

The intersection to be modelled is located along the Albany Expressway corridor that intersects with the Massey University entrance. The intersection was chosen as it represents most of the typical signalised intersection layouts with two give-way left turn slip lanes. Albany Expressway is classified as a regional arterial road on the Auckland regional maps.

The Albany Expressway corridor provides a link between Coatesville and the Diary Flat area of Rodney. In addition, it provides links to the Northern Motorway. The intersection is located just south of the Albany Village and west of Albany Mega Centre. The speed limit along the corridor is 80km/h[1]. The intersection is carrying in excess of 9,000 vehicles per day. Additionally, it has spare capacity and operates well under capacity with degree of saturation of approximately 80%.

Figure 8-1 shows the location of the study intersection highlighted in red.

Figure 8-1 Map of site location

The intersection is coordinated with other signalised intersections along the Albany Expressway corridor. This is to provide progression (green wave) along the corridor to minimise the delays and number of stops.

[1] It is acknowledged that the speed limit of 80km/h is not typically used at signalised intersections; however, it is unlikely to affect the modelling analysis. It only increased the yellow time to 5.5s in the models for the main road approaches, instead of the standard 4s for any intersection with speed limit of 50km/h.

8.3 Model Specifications

The following specifications have been used during the development of the SIDRA base model and subsequent option models in order to obtain comparable results; the values and parameters were fixed as indicated in Table 8-1:

Table 8-1 Model Specifications

Specification	Value
Model Year	2016
Time Period	17:00 to 18:00 PM
Peak Hour Factor	95%
Heavy Vehicles Proportion	5%
Pedestrian Frequency	Called every cycle
Intergreen Times	Obtained from SCATS time settings
Volumes	Obtained from SCATS counts
Cycle Time	Fixed to 120 seconds for base and options models
Delay and Queue Model	SIDRA standard model used
Level Of service	Based on delay (HCM2000)
Intersection Type	Signalised fixed time
Peak Flow Period	30 minutes
Unit Time for Volume	60 minutes
Queue Percentile	95%
Signal Coordination (Arrival Type)	Arrival Type = 4 (Favourable)

8.4 Data Sources

A number of data sources have been referred to during the development of this SIDRA model. These are listed below.

8.4.1 Intersection layout

The intersection layout, lane arrangements and measurements have been obtained from aerial photos via the Auckland GIS viewer map. The length of the turning bays have been measured from the aerial photo. Figure 8-2 shows the aerial photo of the intersection. The left turn slip lanes that have been selected for assessments are highlighted in blue.

It is noted that the two parallel black lines (cross walklines) were added to the university leg because the available aerial photo is outdated by 4-5 years.

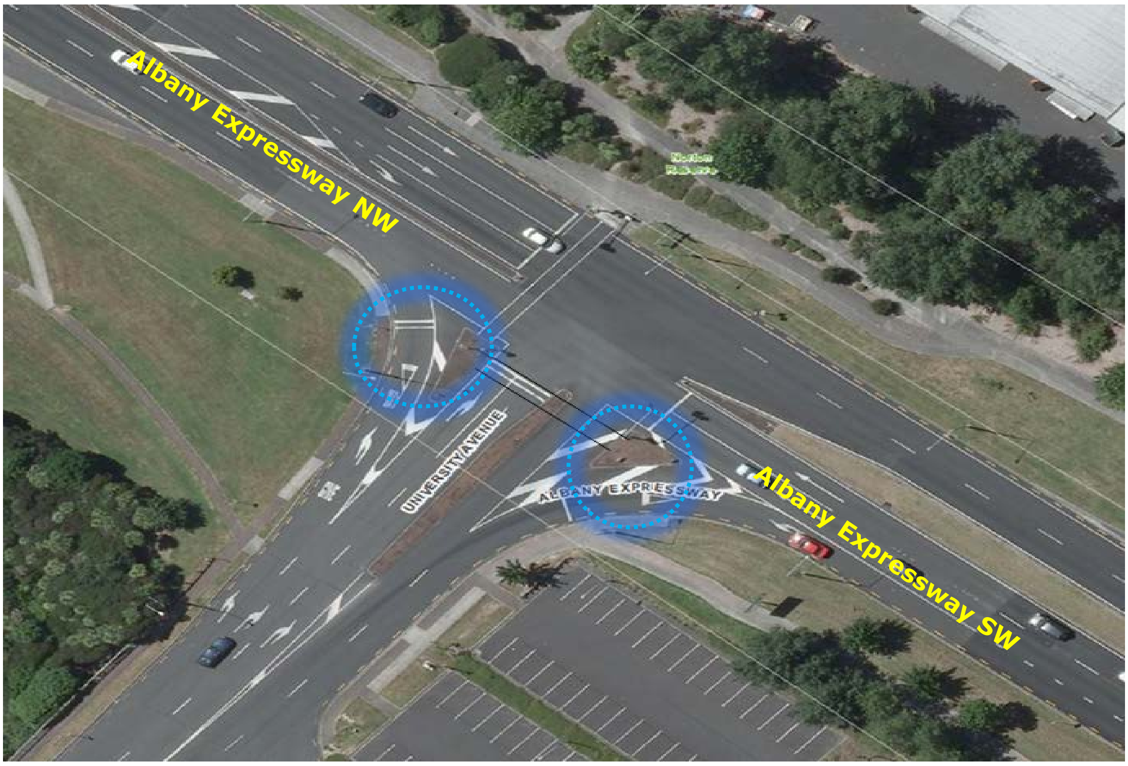


Figure 8-2 Aerial photo of the study site

8.4.2 Traffic volumes

Volumes have been extracted from SCATS data for the date of 15 March 2016. Table 8-2 shows the traffic volumes used for the modelling analysis, from 17:00 to 18:00 pm. It is noted that the morning peak has similar flows to the evening peak. As the left turn into Massey University is high in AM peak, while the left turn flows out of Massey University is high in PM peak.

Since the purpose of this task was to determine the effects of various left turn treatments on the intersection operation; therefore, the peak time chosen was irrelevant to the task. As a result, the PM peak was chosen in the modelling exercise.

Table 8-2 SCATS traffic counts

Approach	Turn	Flows (17:00-18:00PM)
Albany Expressway NW	Through	614
	Right	136
Albany Expressway SW	Through	795
	Left	143
Massey University Avenue	Right	333
	Left	301

8.4.3 Signal phasing and timing

The key operational information relating to this intersection has been directly extracted from SCATS and supplied by ATOC Smales. The SCATS site graphic is provided in Figure 8-3. This is a technical representation of the traffic signal operation for the selected intersection. The following is a clarification of the different key elements used in Figure 8-3:

- The long green arrows in the boxes (the four 4-phases to the left of the image: A, B, C and D) indicate the direction vehicles may travel in within each traffic signal phase;
- The small green double ended arrows in the same boxes indicate the pedestrian movements that operate in each vehicle phase;
- The purple numbers on the intersection diagram represent movements that are allowed to run within a traffic phase; they are termed signal groups; and
- The green boxes containing a number on the intersection diagram represent vehicle detector loops. These detectors pick up where a vehicle is at the stoplines of the intersection and put in a demand for a traffic phase. The blue filled colour means that the detector is occupied, and no colour filled means that the detector is vacated. When the green traffic phase is operational, the detectors also extend the green time for each traffic phase up to a pre-determined maximum (if there is sufficient traffic to make the signals run out to the pre-set maximum); and
- If vehicles are not present on a particular leg of the intersection then the associated traffic phase can be skipped and the next phase can be run.

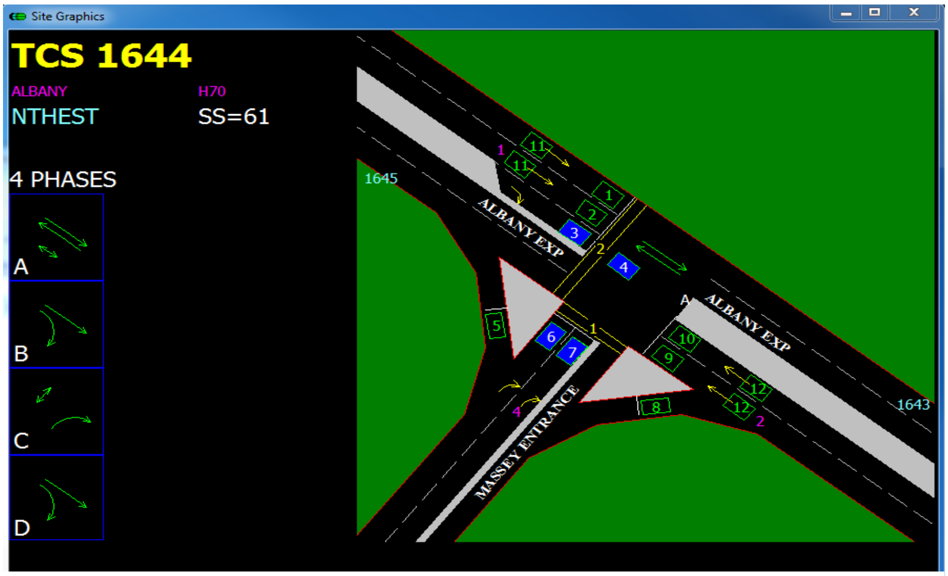


Figure 8-3 TCS1644 – SCATS site graphic

The key operational information relating to this intersection, as directly extracted from SCATS is summarised in Table 8-3. The values in the table represent the average of the data over the PM peak hour (17:00 to 18:00 pm).

Figure 8-4 outlines the details of the signal phasing diagram, including the different traffic and pedestrian phases, left turn slip lane movements (showed with a dotted red arrow) and this should be used when reading the information provided in Table 8-3.

The intersection runs three different traffic phases with a repeat right turn phase which is inactive (B phase). P1 is the pedestrian crossing across Massey University Avenue and P2 is the crossing across Albany Expressway. They run in A phase and C phase respectively.

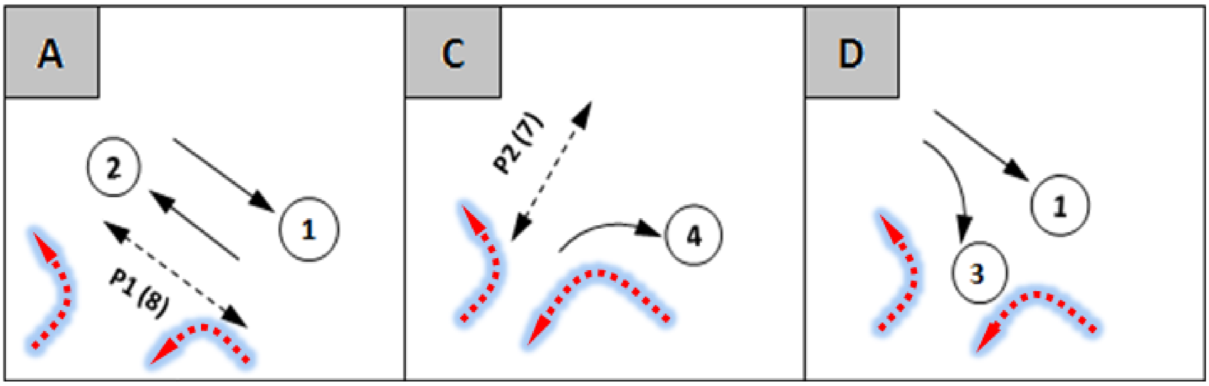


Figure 8-4 Signal phasing diagram indicating the left turn slip lane movements

Table 8-3 Summary of signal phasing and timing for site 1644

Vehicle Phase	**Phase Time Seconds**	**% of Cycle Time**	**Intergreen Times (Yellow+ All-Red) Seconds**
A	74	61.6	6.5
C	25	20.8	5
D	22	18.3	7.5
B	Note that B phase is a repeat right turn phase but not in operation		
Phase Sequence	A-C-D		
Pedestrian Phase	**Walk Time[2] (steady green man display)**	**Clearance Time[3] (flashing red man display)**	
P1	6	16	
P2	6	18	

[2] Walk Time is indicated by a steady green person display (called a "green man") that indicates to the pedestrians that they may commence their crossing, which is typically set to 6 seconds.

[3] Clearance Time is indicated by a flashing red person display. The time is calculated by dividing the maximum crossing distance measured from kerb to kerb by a walking speed of 1.2 m/s.

8.5 Scenarios Modelled

A total of 24 scenarios were modelled in order to cover the four different left turn treatments and a range of vehicle volumes.

Primarily two methods have been used to assess the operational performance of the selected intersection, using the four different left turn treatments. These methods are:

- Method A (MA): Using the existing traffic volumes;
- Method B (MB): Increasing the existing traffic volumes. This method is subdivided into the following two methods:
  - Method B1 (MB1): increasing the left turn traffic flows for the slip lanes; and other intersection movements by two different factors; and
  - Method B2 (MB2): increasing the whole intersection traffic flows.

The details of each method and the scenarios included are explained in the following sections.

8.5.1 Method A: Using the existing traffic volumes

This method uses the existing traffic volumes through the intersection, and it includes eight scenarios. These scenarios are:

Existing Layout

- Base model: give-way left turn slip lanes;

Signal Control

- Option 1: signalised left turn slip lanes;

Exclusive Left Turn Lanes

- Option 2A: exclusive left turn lanes with pedestrian protection for walk time;
- Option 2B: exclusive left turn lanes with pedestrian protection for walk and half clearance intervals;
- Option 2C: exclusive left turn lanes with pedestrian protection for walk and full clearance intervals;

Shared Left Turn Lanes

- Option 3A: shared through and left turn lanes with pedestrian protection for walk time;
- Option 3B: shared through and left turn lanes with pedestrian protection for walk and half clearance intervals; and
- Option 3C: shared through and left turn lanes pedestrian protection for walk and full clearance intervals.

A detail of each scenario is described in the subsequent sections.

8.5.1.1 Base model: give-way left turn slip lanes (existing layout)

The base layout represents the existing situation and operation of the site. The base model was initially calibrated to represent the base line from which other options can be compared against. Two left turn slip lanes were included in the analysis, which are highlighted in a blue colour throughout all scenarios modelled.

Figure 8-5 represents the existing layout for the Albany Expressway and Massey University Avenue intersection.

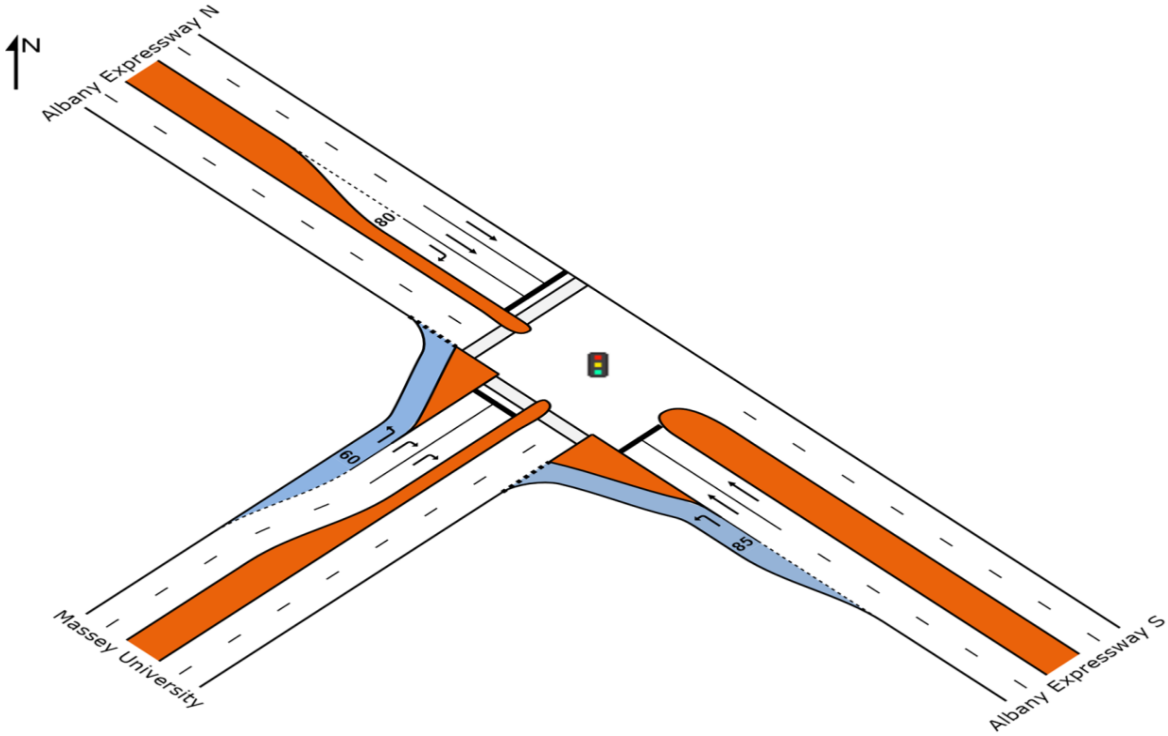


Figure 8-5 Base model-existing layout of Albany Expressway-Massey University Avenue intersection

8.5.1.2 Option 1: signalised left turn slip lanes

This option includes changing the control of both left turn slip lanes from give-way control to signal control, as well as providing a signalised pedestrian crossing at each slip lane as shown in Figure 8-6.

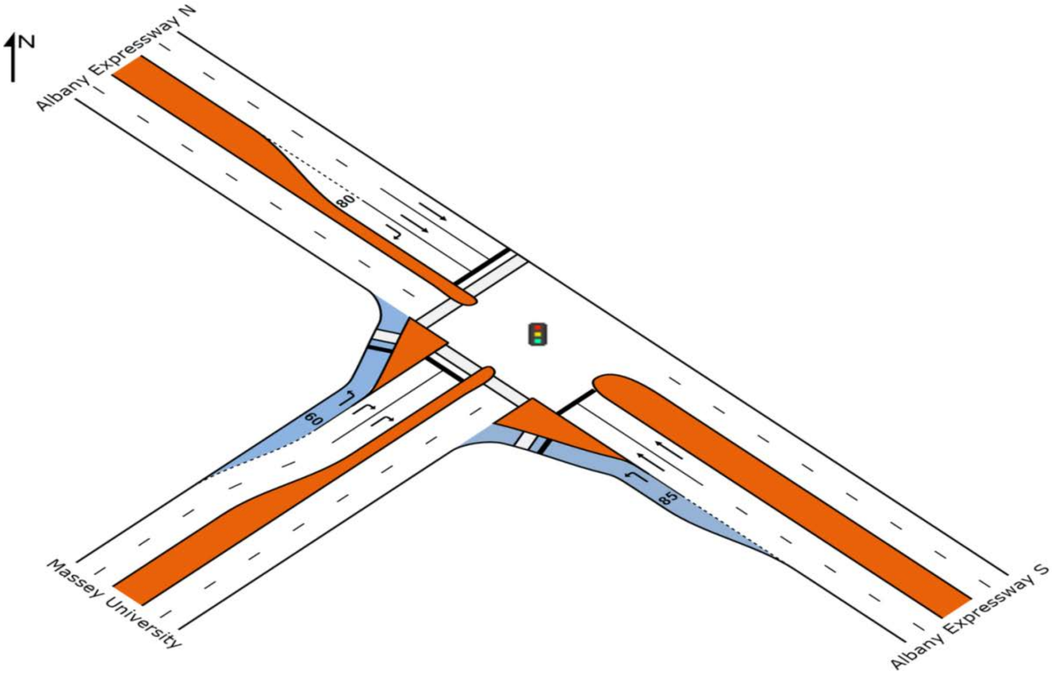


Figure 8-6 Option 1- signalised the left turn slip lanes

8.5.1.3 Replacing the slip lanes with conventional left turn lanes

The two left turn slip lanes were removed and replaced with two different left turn conventional lanes and applying pedestrian protection time as follows:

- Option 2- an exclusive left turn lane with a pedestrian protection; and
- Option 3- a shared through and left turn lane with a pedestrian protection.

In these two options, a turning vehicle movement conflicts with a pedestrian movement. It may be necessary to provide protection for the pedestrians by holding the left turning traffic on a red aspect display (usually an arrow aspect). Therefore, the pedestrian protection time has to be applied to the two conventional left turn lanes types for safety reasons, and to replicate the current practice in Auckland as well.

The degree of protection depends on the severity of the conflict; however, there are three typical levels of protection being applied to the two options:

- Protection for the whole walk interval;
- Protection for the whole walk interval and half of the clearance interval; and
- Protection for both the whole walk interval and the clearance interval (full protection).

The three types of pedestrian protections are graphically represented in Figure 8-7.

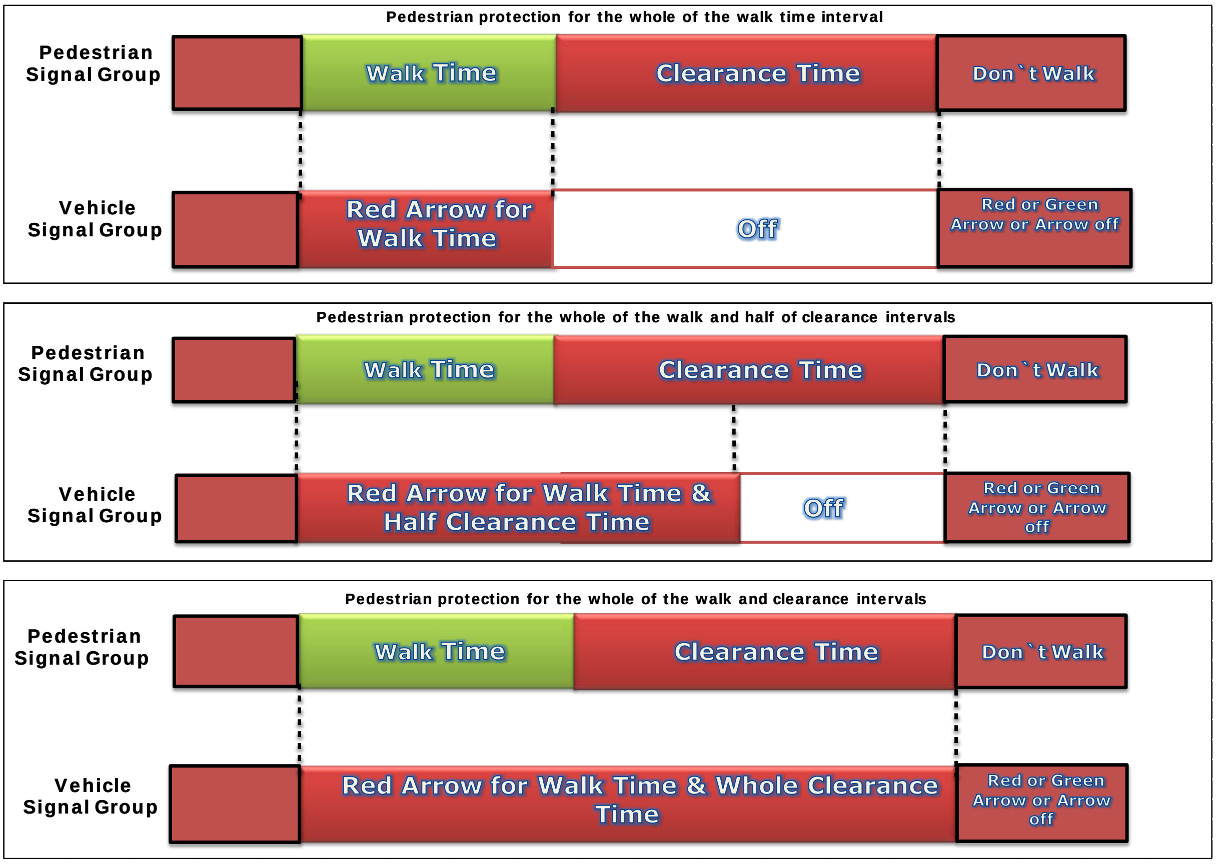


Figure 8-7 Levels of pedestrian protection time

8.5.1.3.1 Protection for the whole walk time interval

When timed protection is used for the whole walk period, vehicles are held on a red signal display (usually an arrow display) for the walk interval, so that the pedestrians can establish their movement and then vehicles are allowed to filter during the clearance interval. For example, in Figure 8-7 the red arrow is held for the Walk time interval (typically 6s) and then changes to “off” to allow vehicles to filter through the pedestrian movement.

8.5.1.3.2 Protection for the whole walk interval and half of clearance interval

When this degree of protection is used, the operation is the same as pedestrian protection for the whole of the walk time, except that the red signal display is held for the walk interval and half of the clearance interval. Then the vehicles are allowed to filter during the second half of the clearance interval. For example, in Figure 8-7 the red arrow is held for the walk time interval and half of the clearance interval then changes to “off” to allow vehicles to filter through the pedestrian movement.

8.5.1.3.3 Protection for the whole walk interval and clearance interval

When full protection is applied, vehicles are held on a red signal display for the entire time of the walk and clearance intervals. As in Figure 8-7, the pedestrian movement is fully protected by holding the vehicle signal group on ‘red’ for the whole of the walk and clearance intervals. Then, the left turning traffic may receive a green arrow but for very short period, depending on the phase time left.

It is worth noting that this protection time can impact on the operational performance of any signalised intersection; therefore, they were included in the analysis.

The three typical pedestrian protection times were applied to the two main options: Option 2 - an exclusive left turn lane and Option 3 - shared through and left turn lane. This is due to the fact that they are the only two situations where the conflict between the turning vehicle and pedestrian movement is applicable.

It is noted that the clearance times for both P1 and P2 were calculated for each option, then the pedestrian protection times was applied accordingly.

8.5.1.4 Option 2A: exclusive left turn lanes with walk protection

In this option, the left turn slip lanes were removed and converted to exclusive conventional left turn lanes, as shown in Figure 8-8. Pedestrian protection time was set to 6s for the walk time only.

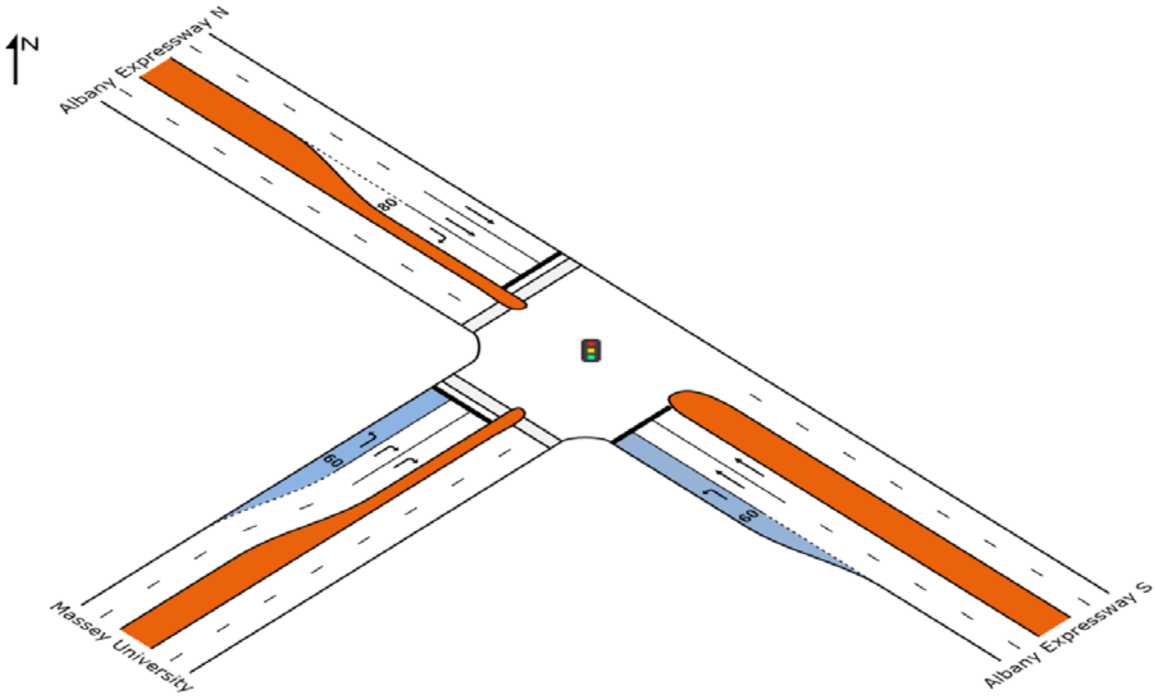


Figure 8-8 Option 2 - left turn slip lanes removed and replaced with an exclusive left turn lanes

Signal phasing remained the same except the overlap phasing was added in the phase sequence to replicate the signal phasing design practice. Figure 8-9 shows the left turn movements (circled in blue) overlapping with the right turn movements in phase C and phase D respectively. Additionally, the left turn movements that conflicts with pedestrian crossings were circled in red (left turning traffic may filter through pedestrians).

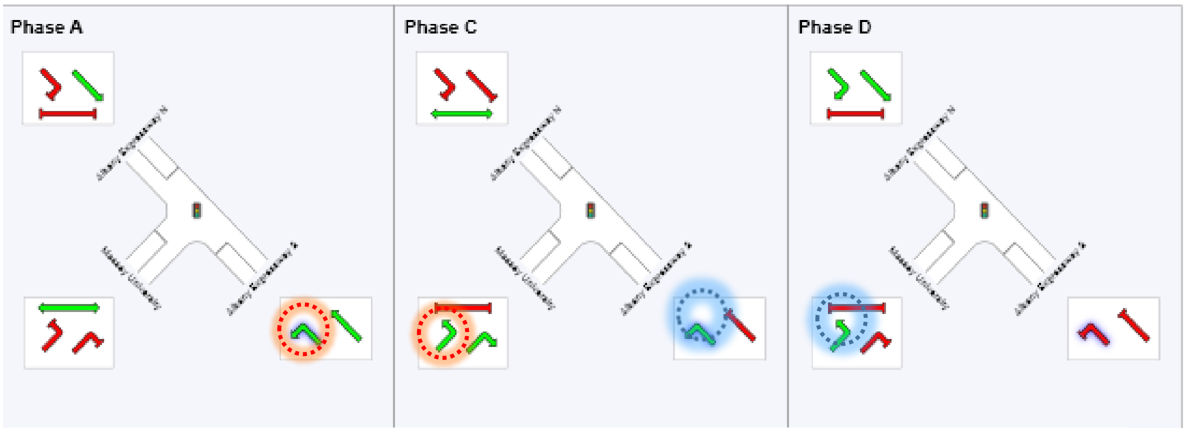


Figure 8-9 Phasing diagram showing the overlap movements

8.5.1.5 Option 2B: exclusive left turn lanes with walk and half clearance protection

This option is the same as Option 2A except that the pedestrian protection time was increased to the walk time and half of the clearance time for both pedestrian crossings: P1 and P2 to 17s and 18s respectively.

8.5.1.6 Option 2C: exclusive left turn lanes with walk and full clearance protection

This option is identical to Option 2A apart from the pedestrian protection time that was recalculated to achieve the protection for the walk and full clearance times. As a result, it increased for P1 and P2 to 28s and 30s respectively;

8.5.1.7 Option 3A: shared left turn lanes with walk protection

This Option 3A is similar to Option 2A but the two left turn slip lanes were removed and replaced by a *shared through and left turn* lane on the SW approach and a *shared right and left lane* on the Massey University approach as indicated in Figure 8-10. Pedestrian protection time was set to 6s for walk time only.

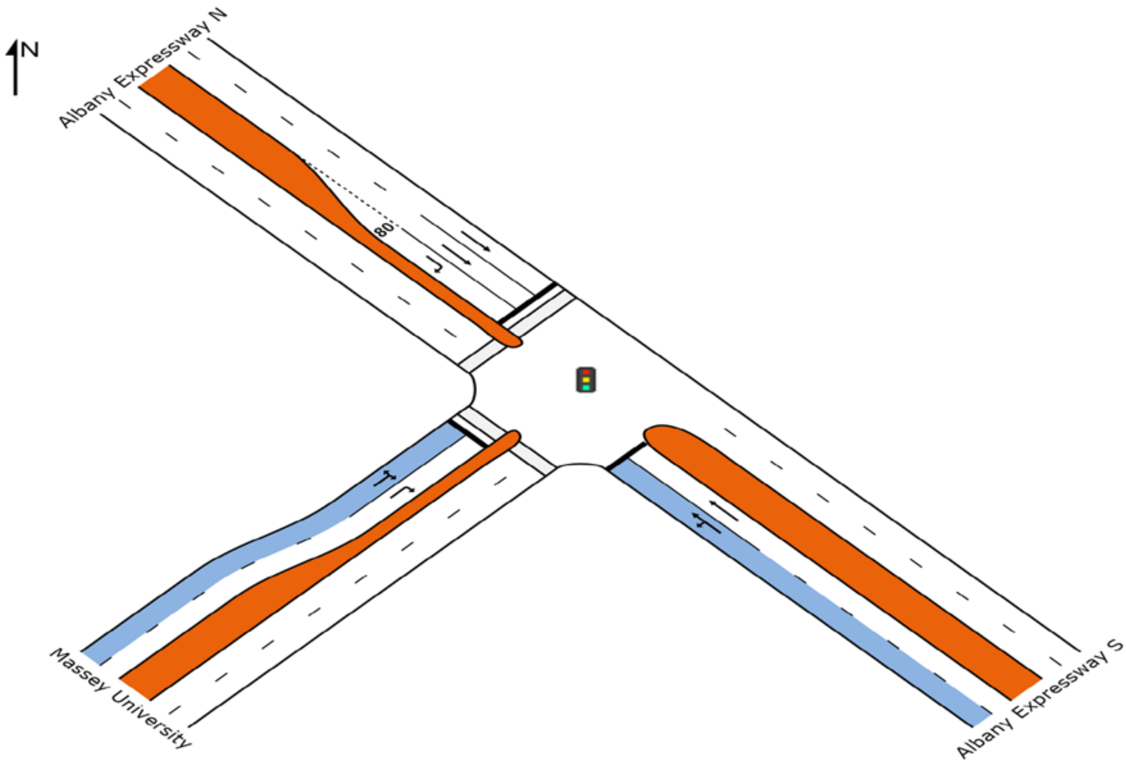


Figure 8-10 Option 3 - left turn slip lanes removed and shared left turn lanes created

8.5.1.8 Option 3B: shared left turn lanes with walk and half clearance protection

This option is the same as Option 3A except the pedestrian protection time was increased for P1 and P2 to 16s and 17s respectively.

8.5.1.9 Option 3C: shared left turn lanes with walk and full clearance protection

This option is similar to Option 3A, but the pedestrian protection time was increased to provide a full pedestrian protection time. Thus, P1 and P2 were increased to 27s and 28s respectively.

It is worth noting that the protection times applied for the above options were incorporated into the SIDRA models as vehicle lost times.

8.5.2 Method B (MB): increasing the existing traffic volumes

This method was mainly a sensitivity test to investigate the effect of traffic volumes on the operational performance of the eight scenarios modelled in Method A. The sensitivity of each scenario was tested by increasing the traffic flows incrementally by a set factor as detailed in the following subsections.

SIDRA was used to plot the average intersection delay, delay for the worst movement, vehicle operating cost, and degree of saturation for each scenario against the flow scale.

SIDRA plots the outcomes of each increment on graphs. These graphs can be compared to see if there are any differences between the base model and each scenario. This is to determine which scenario has a greater impact on the intersection performance as the traffic volumes were increased. Additionally, the graphs allow the identification of the tipping point at which performance indicators begin to increase rapidly.

This method can be subdivided into two key methods:

- Method B1 (MB1): increasing the left turn traffic flows for the slip lanes and other intersection movements; and
- Method B2 (MB2): increasing the whole intersection traffic flows.

The eight scenarios modelled in Method A were used as base models for the following scenarios.

8.5.2.1 Method B1 - increase the left turn traffic volumes

The aim of this method was to increase traffic flows through the intersection by a small factor, and increase the traffic flows of the left turn slip lanes by a larger factor. So, the left turn traffic flows were factored up by an approximately 140% and other intersection movements factored up by 110%. This was to investigate whether these combinations have any effect on the results.

Due to SIDRA limitations, this was achieved by applying a constant flow scale factor (130%) under “volume factor tab” for left turn movements only and applying 110% scale factor under the “demand and sensitivity analysis” for the entire intersection. Then SIDRA multiplied these factors (1.3 x 1.1=1.43) to achieve 143% increase in flows for the left turn movements and 110% for the whole intersection.

8.5.2.2 Method B2 - increase the whole intersection traffic volumes

The objective of this method was to investigate whether varying the whole intersection volume has any effect on the results. In this method, the traffic flows for all movements at the intersection were incrementally factored up to 150% (50% increase).

8.6 Model Calibration

The base model was calibrated by adjusting the saturation flows, other parameters and settings until the modelled queues and signal timings were similar to the observed state, so that the outputs were a suitable representation of observed conditions.

Site observations over the selected PM peak period have been compared against the modelled queues to confirm that the SIDRA model is a suitable representation of the site. This site visit was undertaken on Wednesday 11 May 2016 for the duration of the one hour peak (5pm to 6pm).

Peak observed queues over the hour have been compared to the average maximum modelled queues. While an exact fit is not expected due to difficulties in establishing the speed of moving vehicles from a distance and defining whether a slow moving vehicle is part of a queue, the model outputs should agree with the observations in general terms.

A comparison of site observations against modelled average maximum queues from the PM peak is shown in Table 8-4.

Table 8-4 Comparison of modelled queues and site observations

Approach	Modelled Average Maximum Queue	Observed Average Maximum Queue	% Difference
Albany Expressway NW	9	8	-1
Albany Expressway SW	10	12	+2
Massey University Avenue	10	9	-1

Generally, the data was within acceptable limits when compared to the observed data. As a means of confirming the validity of the SIDRA model, phase timings derived through the modelling were checked against the existing operation data obtained from SCATS, as summarised in Table 8-5.

Table 8-5 Comparison of modelled signal timings and site observations

Vehicle Phase	Phase Time Seconds (Observed)	Phase Time Seconds (Modelled)	% of Cycle Time (Observed)	% of Cycle Time (Modelled)
A	74	71	61	59
C	25	28	21	23
D	22	21	18	18

In summary, the calibration and validation results show that the base SIDRA model accurately replicates the actual traffic conditions and performance of the intersection. It is therefore considered that the base model is fit to assess the impacts of other scenarios.

8.7 Operational Measures

The following performance measures were used to evaluate the intersection operation for each scenario in Method A: Degree of Saturation (DOS), the Control Delay, Level of Service (LOS) and Queue length. In Method B, average delay, delay for the worst movement, vehicle operating cost and DOS were used as performance measures for each scenario.

8.7.1 Degree of Saturation

Degree of Saturation represents the sufficiency of an intersection to accommodate vehicle demand. It is calculated from the volume of traffic divided by the capacity of the approach to an intersection.

8.7.2 Delay and Level of Service (LOS)

Delay is defined in High Capacity Manual 2000 control delay; the average delay is used as the basis for determining LOS. Intersection control delay is generally computed as a weighted average of the average control delay for all lane groups based on the volume within each lane group. Delay thresholds for LOS are given in Table 8-6.

Table 8-6 HCH2000 LOS and delay criteria for signalised intersections

Los	Control Delay per vehicle (sec)
A	<10
B	10 to 20
C	20 to 35
D	35 to 55
E	55 to 80
F	>80

8.7.3 Queue length

Estimates of vehicle queue length for design purposes are typically based on the 95th percentile queue that is expected during the design period. The HCM 2000 provides procedures for calculating the 95th percentile queue. It is defined to be the queue length that has only a 5-percent probability of being exceeded during the analysis time period.

8.8 Model results

This section presents the results of the intersection performance for the 24 modelled scenarios. Detailed descriptions of these scenarios were explained in Section 8.5. These scenarios were assessed in SIDRA and compared against each other. The main eight scenarios are listed as flows:

- Base model: give-way left turn slip lanes;
- Option 1: signalised left turn slip lanes;
- Option 2A: exclusive left turn lanes with pedestrian protection for walk time;
- Option 2B: exclusive left turn lanes with pedestrian protection for walk and half clearance intervals;
- Option 2C: exclusive left turn lanes with pedestrian protection for walk and full clearance intervals;
- Option 3A: shared through and left turn lanes with pedestrian protection for walk time;
- Option 3B: shared through and left turn lanes with pedestrian protection for walk and half clearance intervals; and
- Option 3C: shared through and left turn lanes pedestrian protection for walk and full clearance intervals.

8.8.1 Results of Method A (MA): using the existing traffic volumes

The eight scenarios (base model inclusive) were assessed in SIDRA. The results of each scenario are compared against the base model. The performance indicators provided below are DOS, average delay, LOS and 95% back of queue for each movement. A more detailed result can be found in Appendix F.

8.8.1.1 MA - results of base model

Table 8-7 MA - modelling results for base model

Intersection Approach	Movement	DOS	Avg Delay (sec)	LOS	95% Back of Queue (m)
Albany Expressway SE	Left Lane 1	0.139	7.2	LOS A	9.4
	Through Lane 2	0.469	11.9	LOS B	70.9
	Through Lane 3	0.469	11.9	LOS B	72.9
	Approach	0.469	11.2	LOS B	72.9
Albany Expressway NW	Through Lane 1	0.233	0.8	LOS A	5.9
	Through Lane 2	0.233	0.8	LOS A	5.7
	Right Lane 3	0.866	73.9	LOS E	69.4
	Approach	0.866	14.1	LOS B	69.4
Massey University Avenue SW	Left Lane 1	0.405	9.2	LOS A	40.7
	Right Lane 2	0.728	60.1	LOS E	74.6
	Right Lane 3	0.728	59.9	LOS E	76.9
	Approach	0.728	35.9	LOS D	76.9
Intersection	ALL	0.866	18.9	LOS B	76.9

Table 8-7 shows the model results for the existing layout. It indicates that the intersection operates well under capacity (DOS is 86%), LOS B with minimal delays and queues at all approaches. In addition, the left turn slip lane on SE and SW approaches has minimal delays of 7s and 9s respectively, which is insignificant.

It is worth noting that the average delays of the Right Lane 3 on the NW approach and Right Lane 3 on the SW approach are higher than other approaches. This is due to the intersection being coordinated in such a way that the through movements on Albany Expressway take priority over other movements and receive more green time.

8.8.1.2 MA - results of Option 1

Table 8-8 compares the results of Option 1 with the base model.

Table 8-8 MA - modelling results for Option 1 compared with base model

		Base model				Option 1			
Intersection Approach	**Movement**	**DOS**	**Avg Delay (sec)**	**LOS**	**95% Back of Queue (m)**	**DOS**	**Avg Delay (sec)**	**LOS**	**95% Back of Queue (m)**
Albany Expressway SE	Left Lane 1	0.139	7.2	LOS A	9.4	0.668	58.3	LOS E	63.4
	Through Lane 2	0.469	11.9	LOS B	70.9	0.703	32.5	LOS C	137.6
	Through Lane 3	0.469	11.9	LOS B	72.9	0.703	32.5	LOS C	141.6
	Approach	0.469	11.2	LOS B	72.9	0.703	36.4	LOS D	141.6
Albany Expressway NW	Through Lane 1	0.233	0.8	LOS A	5.9	0.233	0.8	LOS A	5.9
	Through Lane 2	0.233	0.8	LOS A	5.7	0.233	0.8	LOS A	5.7
	Right Lane 3	0.866	73.9	LOS E	69.4	0.322	41.6	LOS D	48.1
	Approach	0.866	14.1	LOS B	69.4	0.322	8.2	LOS A	48.1
Massey University Avenue SW	Left Lane 1	0.405	9.2	LOS A	40.7	0.914	70.8	LOS E	161.1
	Right Lane 2	0.728	60.1	LOS E	74.6	0.728	60.1	LOS E	74.6
	Right Lane 3	0.728	59.9	LOS E	76.9	0.728	59.9	LOS E	76.9
	Approach	0.728	35.9	LOS D	76.9	0.914	65.2	LOS E	161.1
Intersection	ALL	0.866	18.9	LOS B	76.9	0.914	35.2	LOS D	161.1

The key results of the assessment were:

- Signalising the left turn slip lanes reduced the level of service of LT SE approach from LOS A to LOS E; and from LOS D to LOS E for the LT SW approach.
- The average delay for both left turn lanes increased from 7s to 58s and from 9s to 71s respectively, which is substantial.
- The average intersection delay increased to 35s from 19s and queue length increased to 160m from 77m. In addition, the intersection LOS was reduced to D from B.
- This option has pushed the intersection to operate at capacity with a DOS of 91%, as a result of increasing delay and DOS for the left turn lane on the SW approach.
- In summary, results show that there is a significant reduction in the efficiency of intersection operation compared to the base model.

8.8.1.3 MA - results of Option 2A

Table 8-9 compares the results for the base model with Option 2A.

Table 8-9 MA - modelling results for Option 2A compared with base model

		Base model				Option 2A			
Intersection Approach	**Movement**	**DOS**	**Avg Delay (sec)**	**LOS**	**95% Back of Queue (m)**	**DOS**	**Avg Delay (sec)**	**LOS**	**95% Back of Queue (m)**
Albany Expressway SE	Left Lane 1	0.139	7.2	LOS A	9.4	0.159	10.7	LOS B	19.1
	Through Lane 2	0.469	11.9	LOS B	70.9	0.499	14.9	LOS B	82.8
	Through Lane 3	0.469	11.9	LOS B	72.9	0.499	14.9	LOS B	85.2
	Approach	0.469	11.2	LOS B	72.9	0.499	14.3	LOS B	85.2
Albany Expressway NW	Through Lane 1	0.233	0.8	LOS A	5.9	0.244	2.1	LOS A	13.7
	Through Lane 2	0.233	0.8	LOS A	5.7	0.244	2.1	LOS A	13.4
	Right Lane 3	0.866	73.9	LOS E	69.4	0.866	73.9	LOS E	69.4
	Approach	0.866	14.1	LOS B	69.4	0.866	15.1	LOS B	69.4
Massey University Avenue SW	Left Lane 1	0.405	9.2	LOS A	40.7	0.838	55.5	LOS E	139.7
	Right Lane 2	0.728	60.1	LOS E	74.6	0.612	53.3	LOS D	68.7
	Right Lane 3	0.728	59.9	LOS E	76.9	0.612	53.2	LOS D	70.9
	Approach	0.728	35.9	LOS D	76.9	0.838	54.3	LOS D	139.7
Intersection	ALL	0.866	18.9	LOS B	76.9	0.866	25.5	LOS C	139.7

The main findings of the assessment were:

- The average delay for the LT lane SE approach increased slightly in Option 2A from 7s to 11s but it increased considerably from 9s to 55s for the LT lanes SW approach. This is due to firstly, the Albany Expressway is coordinated phase, which receives most of the green times and takes the priority over the side road (Massey University Ave.). Secondly, the left turn traffic volume of this movement is greater than the other left turn movement (301vph vs. 143vph).
- The intersection average delay increased slightly by 6s, LOS was reduced to C and the queue length doubled, and reaching approximately 140m on SW approach.
- The intersection DOS has remained the same around 86%.

8.8.1.4 MA - results of Option 2B

Table 8-10 outlines a summary comparison of base model to Option 2B.

Table 8-10 MA - modelling results for Option 2B compared with base model

		Base model				Option 2B			
Intersection Approach	**Movement**	**DOS**	**Avg Delay (sec)**	**LOS**	**95% Back of Queue (m)**	**DOS**	**Avg Delay (sec)**	**LOS**	**95% Back of Queue (m)**
Albany Expressway SE	Left Lane 1	0.139	7.2	LOS A	9.4	0.192	16.7	LOS B	28.2
	Through Lane 2	0.469	11.9	LOS B	70.9	0.553	20.1	LOS C	100.9
	Through Lane 3	0.469	11.9	LOS B	72.9	0.553	20	LOS C	103.8
	Approach	0.469	11.2	LOS B	72.9	0.553	19.5	LOS B	103.8
Albany Expressway NW	Through Lane 1	0.233	0.8	LOS A	5.9	0.244	2.1	LOS A	13.7
	Through Lane 2	0.233	0.8	LOS A	5.7	0.244	2.1	LOS A	13.4
	Right Lane 3	0.866	73.9	LOS E	69.4	0.592	57.7	LOS E	58.7
	Approach	0.866	14.1	LOS B	69.4	0.592	12.2	LOS B	58.7
Massey University Avenue SW	Left Lane 1	0.405	9.2	LOS A	40.7	0.893	65.9	LOS E	154.8
	Right Lane 2	0.728	60.1	LOS E	74.6	0.612	53.3	LOS D	68.7
	Right Lane 3	0.728	59.9	LOS E	76.9	0.612	53.2	LOS D	70.9
	Approach	0.728	35.9	LOS D	76.9	0.893	59.3	LOS E	154.8
Intersection	ALL	0.866	18.9	LOS B	76.9	0.893	28	LOS C	154.8

The primary findings of assessment were:

- The results show that the exclusive left turn lane on the SW approach appears to have a substantial impact on the capacity of this movement and at the approach level. The average delay increased considerably from 9s to 66s and LOS was reduced to E from A.
- The average delay for the LT lane SE approach nearly doubled to 16s while the LOS changed from A to B. The queue length increased from 9m to 28m.
- The intersection average delay increased from 19s to 28s, LOS was reduced to C from B and DOS increased slightly to an approximately 89%. In addition, the queue length increased from 77m to 155m.
- It is noted that the right turn lanes delay was slightly reduced as a result of SIDRA optimising signal phasing/timings.

8.8.1.5 MA - results of Option 2C

Table 8-11 summarises a comparison of base model to Option 2C.

Table 8-11 MA - modelling results for Option 2C compared with base model

		Base model				Option 2C			
Intersection Approach	**Movement**	**DOS**	**Avg Delay (sec)**	**LOS**	**95% Back of Queue (m)**	**DOS**	**Avg Delay (sec)**	**LOS**	**95% Back of Queue (m)**
Albany Expressway SE	Left Lane 1	0.139	7.2	LOS A	9.4	0.246	25.7	LOS C	38.1
	Through Lane 2	0.469	11.9	LOS B	70.9	0.699	29.8	LOS C	123.5
	Through Lane 3	0.469	11.9	LOS B	72.9	0.699	30.6	LOS C	143.4
	Approach	0.469	11.2	LOS B	72.9	0.699	29.5	LOS C	143.4
Albany Expressway NW	Through Lane 1	0.233	0.8	LOS A	5.9	0.263	4.5	LOS A	25.5
	Through Lane 2	0.233	0.8	LOS A	5.7	0.263	4.5	LOS A	24.8
	Right Lane 3	0.866	73.9	LOS E	69.4	0.489	53.3	LOS D	55.8
	Approach	0.866	14.1	LOS B	69.4	0.489	13.4	LOS B	55.8
Massey University Avenue SW	Left Lane 1	0.405	9.2	LOS A	40.7	0.903	68.6	LOS E	158.5
	Right Lane 2	0.728	60.1	LOS E	74.6	0.493	47.1	LOS D	63.7
	Right Lane 3	0.728	59.9	LOS E	76.9	0.493	47	LOS D	65.8
	Approach	0.728	35.9	LOS D	76.9	0.903	57.3	LOS E	158.5
Intersection	ALL	0.866	18.9	LOS B	76.9	0.903	31.9	LOS C	158.5

Based on Table 8-11, a number of key results were identified:

- The average delay for the LT lane on SE approach increased from 7s to 26s, but it increased considerably from 9s to 67s for the LT lane on SW approach. As a result, the average delay for both approaches increased substantially ranging from 11s to 57s.
- The 95% back of the queue on the SE approach increased from 73m to 143m and similarly it increased from 77m to 158m on the SW approach.
- The LOS was reduced from A to C and from A to E for both LT lanes on the SE and SW approaches respectively.
- The average intersection delay increased from 19s to 32s and DOS increased to 90% which makes the intersection operate at capacity.

8.8.1.6 MA - results of Option 3A

Table 8-12 provides a summary comparison of the base model to Option 3A.

Table 8-12 MA - modelling results for Option 3A compared with base model

		Base model				Option 3A			
Intersection Approach	**Movement**	**DOS**	**Avg Delay (sec)**	**LOS**	**95% Back of Queue (m)**	**DOS**	**Avg Delay (sec)**	**LOS**	**95% Back of Queue (m)**
Albany Expressway SE	Left Lane 1	0.139	7.2	LOS A	9.4	0.672	32.5	LOS C	135.2
	Through Lane 2	0.469	11.9	LOS B	70.9				
	Through Lane 3	0.469	11.9	LOS B	72.9	0.84	34.1	LOS C	221.1
	Approach	0.469	11.2	LOS B	72.9	0.84	33.4	LOS C	221.1
Albany Expressway NW	Through Lane 1	0.233	0.8	LOS A	5.9	0.285	7.6	LOS A	37.3
	Through Lane 2	0.233	0.8	LOS A	5.7	0.285	7.6	LOS A	36.3
	Right Lane 3	0.866	73.9	LOS E	69.4	0.866	73.8	LOS E	69.4
	Approach	0.866	14.1	LOS B	69.4	0.866	19.6	LOS B	69.4
Massey University Avenue SW	Left Lane 1	0.405	9.2	LOS A	40.7	0.907	71.8	LOS E	163.6
	Right Lane 2	0.728	60.1	LOS E	74.6				
	Right Lane 3	0.728	59.9	LOS E	76.9	0.813	53.2	LOS D	152.5
	Approach	0.728	35.9	LOS D	76.9	0.907	62	LOS E	163.6
Intersection	ALL	0.866	18.9	LOS B	76.9	0.907	36.8	LOS D	221.1

The main findings of the assessment were:

- The shared right and left turn lane on the SW approach has an extensive effect on the capacity of this movement and at the approach level. The average delay increased substantially from 36s to 62s and LOS was reduced to E from D.
- The average delay for the SE approach nearly tripled to 33s while LOS was reduced to C.
- The queue length was increased from 70m to 221m on the SE approach, due to the shared lane configuration.
- The intersection delay increased from 19s to 37s, queue length from 77m to 221m and LOS was reduced from B to D.
- It is worth noting that the closing of left turn lanes and the replacement with shared lanes configuration contributed to the deterioration of the intersection performance.

8.8.1.7 MA - results of Option 3B

Table 8-13 outlines a comparison summary between the base model and Option 3B.

Table 8-13 MA - modelling results for Option 3B compared with base model

		Base model				Option 3B			
Intersection Approach	**Movement**	**DOS**	**Avg Delay (sec)**	**LOS**	**95% Back of Queue (m)**	**DOS**	**Avg Delay (sec)**	**LOS**	**95% Back of Queue (m)**
Albany Expressway SE	Left Lane 1	0.139	7.2	LOS A	9.4	0.714	38.5	LOS D	134.6
	Through Lane 2	0.469	11.9	LOS B	70.9				
	Through Lane 3	0.469	11.9	LOS B	72.9	0.893	40.9	LOS D	266.3
	Approach	0.469	11.2	LOS B	72.9	0.893	40	LOS D	266.3
Albany Expressway NW	Through Lane 1	0.233	0.8	LOS A	5.9	0.298	9.3	LOS A	43.3
	Through Lane 2	0.233	0.8	LOS A	5.7	0.298	9.3	LOS A	42.1
	Right Lane 3	0.866	73.9	LOS E	69.4	1.125	199.3	LOS F	121.9
	Approach	0.866	14.1	LOS B	69.4	1.125	43.8	LOS D	121.9
Massey University Avenue SW	Left Lane 1	0.405	9.2	LOS A	40.7	1.082	164.6	LOS F	252.6
	Right Lane 2	0.728	60.1	LOS E	74.6				
	Right Lane 3	0.728	59.9	LOS E	76.9	0.752	46.3	LOS D	139.4
	Approach	0.728	35.9	LOS D	76.9	1.082	102.5	LOS F	252.6
Intersection	ALL	0.866	18.9	LOS B	76.9	1.125	58.3	LOS E	266.3

The key results of the assessment were:

- The average delay for SE approach increased from 11s to 40s; on the other hand, it increased significantly from 36s to 102s for the SW approach. Similarly, the average delay for the NW approach increased from 14s to 44s, which is substantial.
- The LOS reduced to D, D and F for the SE, NW and SW approaches respectively.
- The overall intersection delay increased from 19s to 58s, LOS was reduced from B to E, and DOS escalated up to 112%, in which the intersection will operate over capacity.

8.8.1.8 MA - results of Option 3C

Table 8-14 shows the comparison of results for the base model with Option 3C.

Table 8-14 MA - modelling results for Option 3C compared with base model

		Base model				Option 3C			
Intersection Approach	**Movement**	**DOS**	**Avg Delay (sec)**	**LOS**	**95% Back of Queue (m)**	**DOS**	**Avg Delay (sec)**	**LOS**	**95% Back of Queue (m)**
Albany Expressway SE	Left Lane 1	0.139	7.2	LOS A	9.4	0.717	45.5	LOS D	119.5
	Through Lane 2	0.469	11.9	LOS B	70.9				
	Through Lane 3	0.469	11.9	LOS B	72.9	0.896	36.9	LOS D	287.7
	Approach	0.469	11.2	LOS B	72.9	0.896	39.6	LOS D	287.7
Albany Expressway NW	Through Lane 1	0.233	0.8	LOS A	5.9	0.294	8.7	LOS A	41.3
	Through Lane 2	0.233	0.8	LOS A	5.7	0.294	8.7	LOS A	40.2
	Right Lane 3	0.866	73.9	LOS E	69.4	1.876	868.9	LOS F	268.7
	Approach	0.866	14.1	LOS B	69.4	1.876	164.9	LOS F	268.7
Massey University Avenue SW	Left Lane 1	0.405	9.2	LOS A	40.7	2.009	988.6	LOS F	627.1
	Right Lane 2	0.728	60.1	LOS E	74.6				
	Right Lane 3	0.728	59.9	LOS E	76.9	0.772	48.3	LOS D	143.4
	Approach	0.728	35.9	LOS D	76.9	2.009	494.8	LOS F	627.1
Intersection	ALL	0.866	18.9	LOS B	76.9	2.009	204.4	LOS F	627.1

The primary findings of assessment were:

- The average delay for the SE, NW, and SW approaches increased significantly to 40s, 165s and 204s respectively. Likewise, the 95% back of the queue for these approaches extended to 288m, 268m, and 627m respectively.
- The overall intersection average delay increased from 19s to 204s, LOS was reduced to F from B and DOS increased to approximately 200%, in which the intersection operation begins to break down.

8.8.1.9 Discussions - results of Method A

Overall the results for Method A shows that the removal of left turn slip lanes including signalised slip lane scenarios has a significant impact on the intersection efficiency. It increased the delay, queue length and degree of saturation. These effects were also magnified by increasing pedestrian protection times.

In practice, to compensate for these impacts, the cycle time could be increased. However, this will be at the expense of increasing the delay for this intersection and other coordinated intersections. Dissimilar to the base model, the intersection operates more efficiently with spare capacity and less delay compared with other scenarios.

Figure 8-11 illustrates the average delay comparisons for each option for all approaches including the overall intersection.

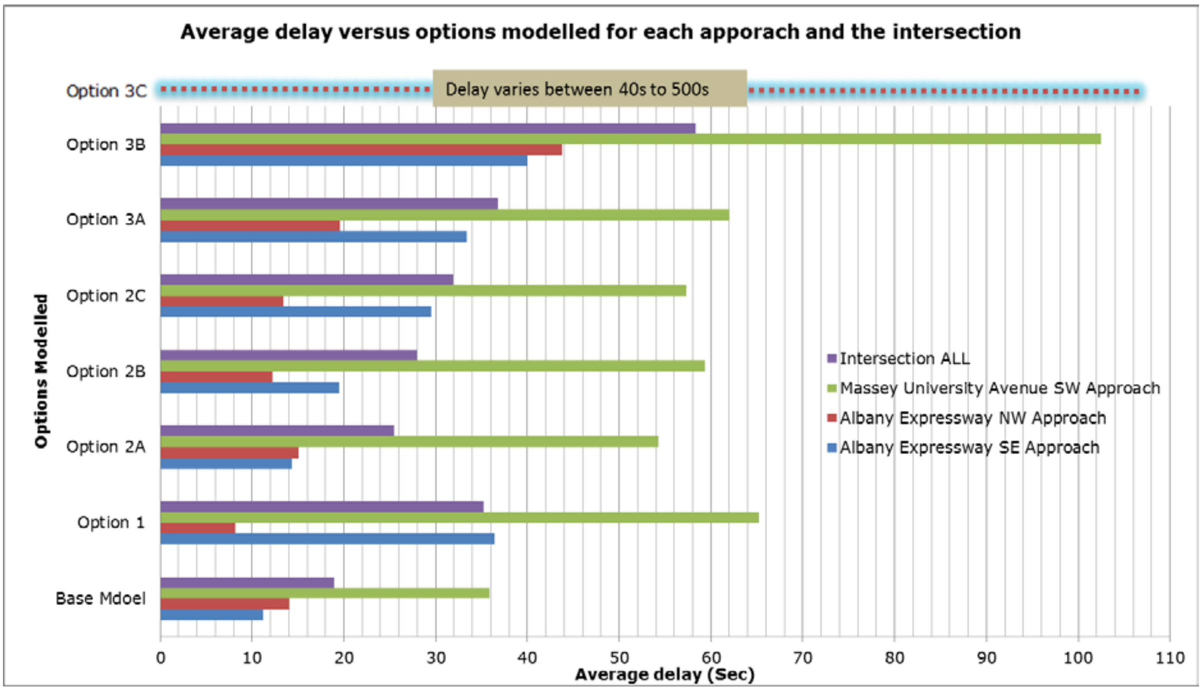


Figure 8-11 Average delay versus options modelled for each approach and the overall intersection

Figure 8-11 shows considerable increase of delays for left turners especially on the SE and SW approaches as well as for the whole intersection. The delay, particularly for Options 2 and 3 was amplified by the increase of the pedestrian protection time. Nevertheless, the effect on Options 3B and 3C was more dramatic.

Additionally, the analysis shows that Options 1 and 2A have similar delays. This suggests that signalising a left turn slip lane is less efficient than having a left turn exclusive lane.

It is evident from Figure 8-11 that there is a consistent pattern throughout all the options of a substantial increase in the delay of left turn approaches, particularly the SW approach.

The main conclusion is the removal of left turn slip lanes coupled with pedestrian protections reduced the intersection efficiency considerably, although the intersection operates under capacity and without any increase in traffic flow.

8.8.2 Results of Method B (MB): increasing the traffic volumes

The eight scenarios in Method A were used as base models for this method. These scenarios were assessed in SIDRA and compared against each other. Intersection performance indicators such as average intersection delay for the worst movement, vehicle operating cost and DOS were analysed for each scenario. However, the average intersection delay is the only performance indicator presented in the main body of the report, but other indicators, SIDRA detailed table results, and graphs can be found in Appendix G and Appendix H.

8.8.2.1 Results of MB1-intersection average delay

Result of the flow scale versus the average intersection delays is shown in Figure 8-12.

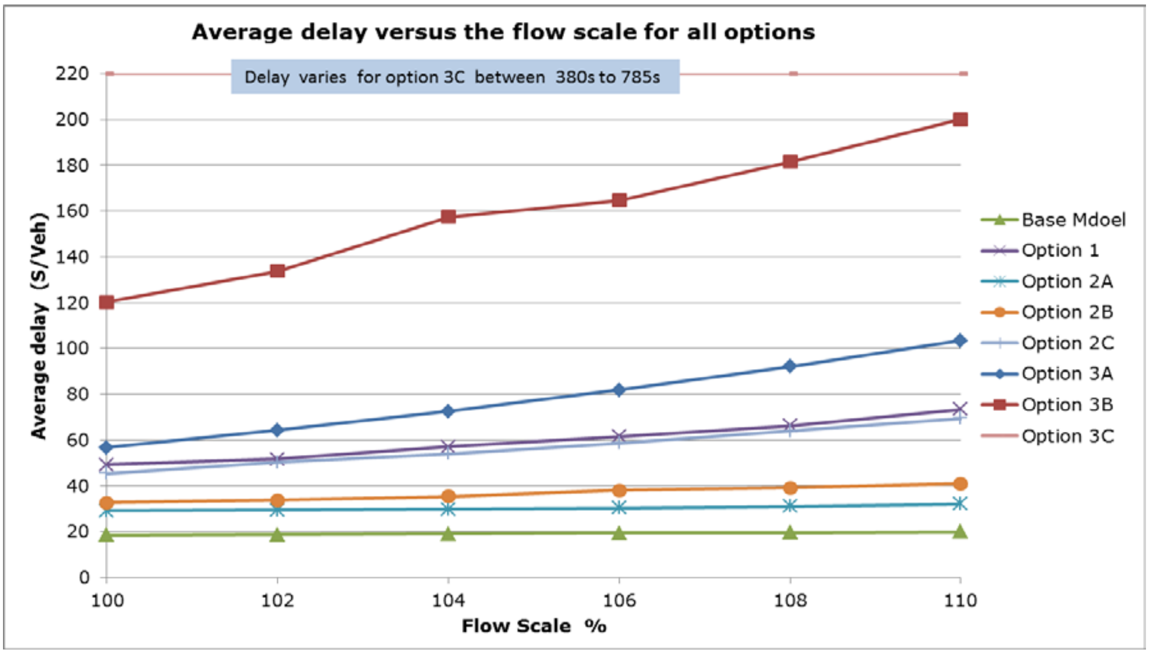


Figure 8-12 MB1- average delay comparison of all options

The results show that the average delays for Option 3C and Option 3B escalate rapidly at 102% of traffic flow scale. The average delay for the base model remains similar as the traffic flow increases. The average delay for both Options 1 and 2C are similar to each other and escalate slightly around 104% of the traffic flow scale.

In summary, the results indicate that as traffic flow increases, there is a minimal change in the intersection performance for the base model, but noticeable change for Options 2A and 2B. Other Options had considerable detrimental effects in the intersection operational performance.

In addition, the flow scale against each of the following performance indicators - the vehicle operating cost, the delay for the worst movement and the degree of saturation - were analysed. This analysis showed similar trends as above.

The results are presented in Appendix G.1, Appendix G.2, and Appendix G.3 respectively.

8.8.2.2 MB2 - intersection average delay

Figure 8-13 shows the average intersection delay versus the flow scale for each option.

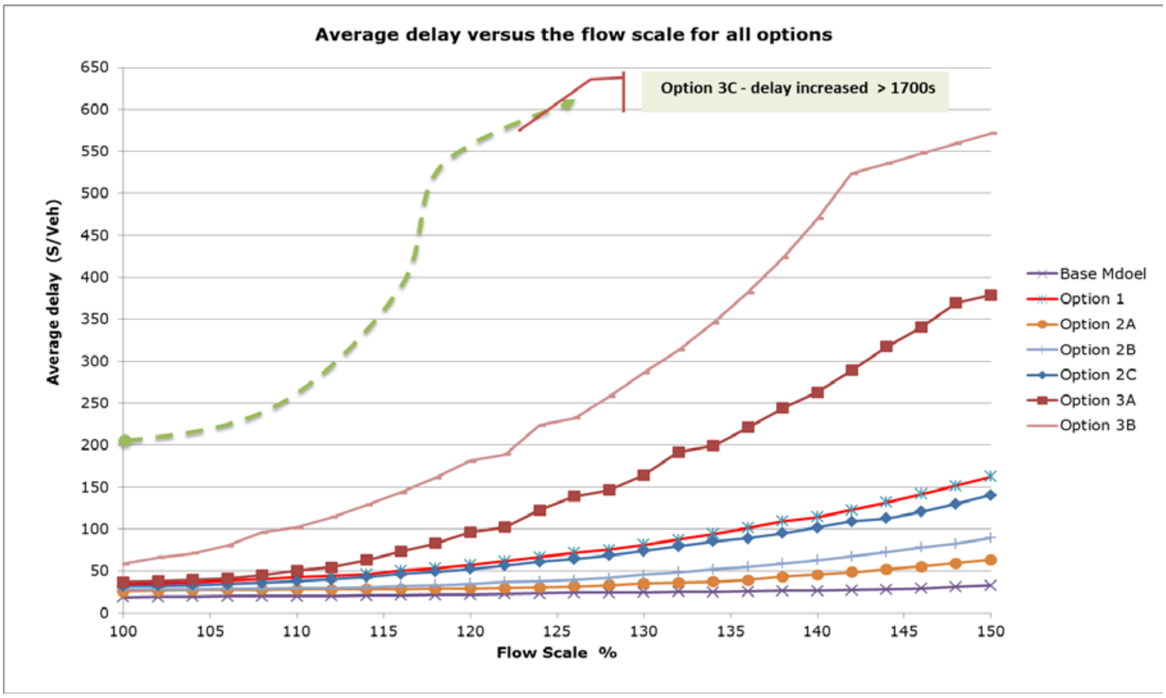


Figure 8-13 MB2 - average delay comparison of all options

The results indicate the average delay for Option 3C exacerbates faster than other Options and at 106% the delay is at approximately 300s. At this volume level the intersection operation breaks down.

In Option 3B, the delay increases exponentially to reach 100s at 110% of the flow scale. While Option 3A reached 100s at 122% of the flow scale. The delays for both Options 1 and 2C is almost identical for up to 140% of the flow scale. The delays for the base model, Option 2A and Option 2B increase proportionally with increasing the flow scale; nevertheless, they remain under 100s.

Furthermore, analysis of the flow scale against each of the following performance indicators - the vehicle operating cost, the delay for the worst movement and the degree of saturation - were conducted to see if there are any pattern difference. The results of the analysis showed similar trends as above and can be found in Appendix H.1, Appendix H.2 and Appendix H.3 respectively.

8.8.2.3 Discussion - results of Method B

It is evident from Figure 8-12 and Figure 8-13 that there is a consistent pattern of a considerable increase in delays in Options 1, 2 and 3 as the flow scale increases, nevertheless, it is more dramatic increase for Options 3B and 3C.

For Method B1, the intersection operation begins to break down at 110% of the flow scale level for Options 3A, 3B and 3C with the delays increasing in a range from 1.7min to 13min.This is where the intersection suffers from an inadequate length of left turn lanes and consequently poor operation of approaches begins to arise.

Likewise for Method B2, in both Options 3B and 3C, the intersection operation begins to break down at 110% of the flow scale level, as a result of the increased delays to 1.7min and 5.7min respectively.

Conversely, the base model with left turn slip lanes performed adequately at all increased levels of the flow scale of the left turn flows and /or intersection flows.

8.9 Summary of results

The 24 scenarios were modelled and analysed employing two primary methods: the first Method A used the existing traffic volumes and the second, Method B, scaled up the intersection flows by a range of factors. The key results of the modelling scenarios are summarised as follows:

- ❖ The analysis highlighted that the removal of the two left turn slip lanes contributed tremendously to the increase of delay experienced by these movements, to their relevant approaches and to the intersection as a whole.
- ❖ In the base model, within both methods, the intersection average delay increased insignificantly from 19s to 33s even with increasing the flow scale. Dissimilar to other options modelled, the intersection average delay increased in range from 26s to 1769s which is a dramatic increase;
- ❖ It is evident from the base model that left turn slip lanes contribute to the resilience of the intersection performance, even with increasing the flow scale of left turn movements and/or the whole intersection;
- ❖ The use of left turn slip lanes can significantly reduce delays experienced by left turning traffic movements, to their relevant approaches and the overall intersection delay. This was evident in both methods; with and without volume increase;
- ❖ The intersection operation for both exclusive and shared left turn lanes scenarios are affected negatively by increasing pedestrian protection time:

- o However, the shared left turn options are worse than the exclusive left turn options (it agrees with Potts et al., 2011); and
- o Pedestrian protection time has a considerable effect on the delay of the left turning traffic; and
- o Pedestrian protection time coupled with shared left turn lane scenarios contributed to an increase in the delay dramatically.

In summary, despite the intersection operating well under capacity for the existing layout, removal of the left turn slip lanes has impacted negatively the intersection performance substantially. In addition, increasing pedestrian protection times has also led to detrimental effects on the intersection efficiency.

The analysis suggests the following points should be considered in the design of a new or a modified signalised intersection:

- ❖ Inclusion of left turn slip lanes where possible as they would contribute immensely to the resilience of intersection performance in the future. This is from the intersection efficiency perspective;
- ❖ Before removing the turn slip lane a careful investigation with robust modelling analysis, should be undertaken considering the following:
    - o Validation and calibration of the intersection model;
    - o Applying pedestrian protection times, and
    - o A sensitivity test should be carried out with various flow factors to determine the likely outcome of intersection operation at the present and in the future.
- ❖ If the decision is made to close off the slip lanes, then it is recommended to:
    - o Replace them with exclusive left turn lanes rather than shared lanes; and
    - o Limit the pedestrian protection time to walk time only, unless there is a significant safety issue that needs to be considered when applying the protection time (should be assessed based on case by case).

Finally, it is recommended for future research, to choose a four legs intersection(s), with a speed limit of 50k/hr, four left turn slip lanes, and operating at capacity. Then to model the effect of removing each of the left turn slip lanes individually and collectively on the intersection performance.

9 Conclusions and Recommendations

Left turn treatments have been considered in the design of signalised intersections by most transportation engineers, especially left turn slip lanes. However, the safety and efficiency of these treatments are largely unknown. Also, there was no crash historical data or established methodology available to evaluate the safety performance and operational performance of the left turn slip lanes. Based upon this, the main key research objectives of this research were raised.

The first objective was to evaluate the safety performance of different left turn treatments at signalised intersections. Initially, the overall crash analysis was conducted. It compared frequency, severity, crash movement codes and contributing factors of crashes involving left turn movements that occurred at various left turn treatments, in the entire network for a period of one year. Secondary to this, a detailed crash analysis was carried out to evaluate in depth the frequency and severity of left turn crashes at selected signalised intersections for the various left turn treatments. Lastly, additional detailed pedestrian crash analysis for the entire signals network. Statistical analysis, using Chi-square test, was conducted to verify the key results.

The second key research objective was to evaluate the operational performance of the various left turn treatments. This was carried out using intersection modelling: in terms of delay, queue length and level of service. As well as conducting further analysis by testing a different range of left turn flows, intersection flows, and pedestrian protection times. This was to determine the wider implications on the intersection operational performance, as a result of increasing traffic flows or pedestrian protection times.

To conduct the safety performance research, the main data collection used involved two types of data. First, signalised intersection data, which included categorising the whole 625 signalised intersections in the network into 1818 approaches that permit left turns. Secondly, the crash data was obtained from CAS for all reported crashes for the 625 signalised intersections, over the period 2010–2014 (inclusive). Then, crashes involving left turn movements were identified and assigned to each of the relevant approaches.

The data collection for this research was a challenging and time consuming task. It took approximately a year, to prepare both the intersection and the crash data sets for analysis. This is due to the limitations of the data availability in the RCA systems as well as CAS data. As a result, extensive manual interrogation was required to gather such data.

The following is a summary of the overall crash analysis, detailed crash analysis, additional detailed pedestrian crash analysis, and the intersection operational performance analysis.

9.1 Overall crash analysis

It was impractical to conduct the overall crash analysis for the five year period due to the large number of crash records and the extensive manual interrogations required. Consequently, only one year was analysed. Data was filtered and narrowed down to 230 left turn crashes which were analysed mainly using two methods: the unclassified left turn crashes and the classified left turn crashes.

The unclassified left turn crashes analysis showed that the vast majority of left turn crashes, 86%, were non-injury crashes. It is worth noting that pedestrian crashes were not a major issue as they represented only 7 crashes, 3%, out of 230 crashes which is not substantial.

The analysis showed that 70% of the predominately left turn crashes were rear-end/obstruction type. It was found that, the contributing factors for left turn crashes were incorrect lane position (34%) and poor observation (24%). On the other hand, the main three left turn crash frequency by object struck involved a post or pole (25%), traffic island (18%) and traffic sign (18%).

The classified left turn crashes analysis revealed that left turn slip lanes experienced 69% of the crashes at 41% of the sites, compared to left turn conventional lanes experiencing 31% of the crashes at 59% of the sites.

In terms of injury crashes, the left turn slip lanes experienced a slightly lower proportion of injuries (61%) to their frequency treatment, and 69% in the signal network. While conventional lanes appeared to experience higher occurrence of injury crashes (39%) relative to their treatments frequency (31%).

Of the slip lanes, the give-way experienced the highest proportion of injury crashes. On the other hand, the shared lane experienced the greatest proportion of injuries for conventional lanes.

It is worth noting that the analysis showed in terms of left turn crash code type, the majority of slip lanes crashes were FB type (15), while the FE type (38) crashes were mainly associated with conventional lanes.

9.2 Detailed crash analysis

For the detailed crash analysis, 84 signalised intersections including 267 approaches that allow left turns were selected and analysed. The 267 left turn approaches were classified according to the different left turn treatment types. Out of these approaches, there were 141 left turn conventional lanes and 126 left turn slip lanes.

The left turn crash data was collected for the five year period. Traffic volume for various left turn approaches, as well as pedestrian activation data were collected from SCATS.

Left turn crashes (including all road user crashes) occurring at the different left turn treatments were compared with their relevant frequency exposure and volume exposure. It was found that in the frequency exposure method, the left turn conventional lanes have slightly lower crash rate than the left turn slip lane.

Contrary to this, in the volume exposure method, the left turn conventional lanes have slightly higher crash rates than the left turn slip lanes. However, the differences in the crash rates between the left turn slip lanes and the left turn conventional lanes (in each method) were minimal.

Hence it is reasonable to conclude that both left turn slip lanes and left turn conventional lanes have a comparable safety performance that was statistically proven.

Additionally, in both exposure methods, the crash rate was different at each of the left turn treatments. Therefore, this suggests that the different types of left turn treatments may differ in their safety performance.

Similar to the overall crash analysis, the proportion of pedestrian crashes to the overall number of crashes was negligible; however, most of these crashes occurred at shared left turn conventional lanes.

9.3 Additional pedestrian crash analysis

Additional pedestrian crash analysis was done on 625 signalised sites comprising 1818 approaches that enable left turns. There was a total of 46 left turn vehicle versus pedestrian crashes occurring at signalised sites. Pedestrian crashes occurring in relation to different left turn treatments in comparison to their treatment frequency are summarised as follows:

- ❖ The proportion of left turn crashes which occurred at various slip lane types compared to their frequency treatments are:
    - o 26% versus 9% for zebra crossing slip lanes;
    - o 15% versus 25% for give-way slip lanes; and
    - o 2% versus 4% for signalised slip lanes
- ❖ The proportion of left turn crashes which occurred at the two conventional lanes types compared to their frequency treatments are as follows:
    - o 39 % versus 38% for shared conventional lanes; and
    - o 17% versus 21 % for exclusive conventional lanes.

The overall proportion of left turn pedestrian crashes was very small compared with their relevant treatments frequency, resulting in a crash rate of 2.35%. The crash rate for the slip lanes differed slightly from the conventional lanes by approximately 0.3%. Hence, they both have a similar safety performance without taking into account pedestrian volume exposure and key design features of each treatment. Additionally, the results highlighted that the zebra crossing slip lanes performed poorly in terms of pedestrian safety. These results were statistically established.

Further analysis on left turn pedestrian crashes was carried out in terms of severity. It was found that the greatest proportion of left turn pedestrian crashes resulted in minor injuries, followed by non-injuries, and a small proportion were serious injuries while no fatal were crashes were recorded.

Left turn slip lanes experienced fewer pedestrian injury crashes than left turn conventional lanes. The largest proportion of pedestrian injury crashes occurred, predominantly at shared conventional lanes and at zebra crossing slip lanes, thus both performed poorly in terms of pedestrian safety.

On analysing the crash data by movement codes, it was found that the pedestrian crashes occurring at conventional lanes were dominated by NC type, while at slip lanes they were overwhelmingly predominated by NA type.

There are safety concerns about the need to cross uncontrolled left turn slip lane movements. These concerns are mitigated by the use of best practice design features including the use of high entry angle, adequate island size and placing zebra crossing on a raised platform. The influence of implementing these design features on the safety performance should be explored in a further research.

Moreover, a comparison study was conducted between the O'Brien et al. (2010) study (the Melbourne study) and the additional pedestrian crash analysis in this research. In both studies, the proportion of left turn crashes involving pedestrians was generally too small compared to their left turn approaches.

Both studies had similar results regarding the proportion of pedestrian crashes occurring at signalised left turn slip lanes as well as in the exclusive left turn conventional lanes. The shared lanes in the two studies experienced higher pedestrian crashes than their frequency in the network. In summary, the Melbourne study indicated that the left turn slip lanes are safer than the left turn conventional lanes for pedestrians; on the contrary, to the present research where both have similar safety performance, which was statistically confirmed.

Due to data and resources limitations in this research, other factors that may influence the safety of left turn treatments type were not investigated. One of these factors is the geometry design of the slip lanes, which can result in some treatments being safer than others. Other factors include: approach gradient, pedestrian and traffic demands, and intervisibility. For example, traffic and pedestrian demands usually govern the use of different left turn treatments type. Hence, the pedestrian exposure can increase at certain treatments rather than others. Again all these factors were not considered due to the limitations of data and resources and should be explored in details in future research.

9.4 Intersection Operational Performance

The 24 scenarios were modelled to assess the intersection performance using two key methods. A method using the existing traffic volumes and the other using scaling up the left turn movements flows and/or intersection flows by a range of factors.

- ❖ The analysis highlighted that the removal of the two left turn slip lanes contributed tremendously to the increase of the delay experienced by these movements, to their relevant approaches and to the overall intersection delay.
- ❖ The two left turn slip lanes, at the modelled intersection, contribute to the resilience of the intersection performance, even with increasing the flow scale of the left turn movements and/or of the whole intersection. The use of left turn slip lanes can significantly reduce delays experienced by left turning traffic movements, to their relevant approaches and the overall intersection delay;
- ❖ The intersection operation for both exclusive and shared left turn lanes scenarios are affected negatively by increasing pedestrian protection time:
    - o The shared left turn options were worse than the exclusive left turn options (it agrees with Potts et al., 2011); and
    - o Pedestrian protection time has a considerable effect on the delay of the left turning traffic; and
    - o Pedestrian protection time coupled with shared left turn lane scenarios contributed to the increase in the delay dramatically.

9.5 Recommendation

It is evident from the analysis that the frequency of left turn crashes was minimal and the greatest proportion of the crashes was non-injury. Also, the safety performance of left turn slip lanes and left turn conventional lanes were similar. However, the left turn slip lanes experienced less injury crashes than the left turn conventional lanes for pedestrians. Furthermore, the left turn slip lane showed greatest efficiency benefits to the intersection operation. Therefore, it is recommended that the following points be considered at signalised intersections:

- ❖ Inclusion of left turn slip lanes into the design of new/modified intersections where possible, as they would contribute immensely to the resilience of the intersection performance in the future;

- Retain the exiting left turn slip lanes where possible. However, if the decision is made initially to investigate the removal of the slip lane, careful consideration with robust modelling analysis should be conducted. This includes validation and calibration, applying pedestrian protection times, and a sensitivity test. Moreover, the decision should be based on data evidence such as left turn crashes involving pedestrians (by sighting TCRs), the island size and other key significant issues;
- If the decision is made to close off the slip lanes, then it is recommended to replace them with exclusive left turn lanes rather than shared lanes. Also, the pedestrian protection time should be limited to walk time only, unless there is a significant safety issue that needs to be considered; and
- In designing signalised intersections, it is recommended to avoid using zebra crossing slip lanes and shared conventional lanes as they performed poorly in terms of pedestrian safety.

The following recommendations for RCA and CAS system

- The RCA should develop and maintain a database that contains records for left turn lane treatments at signalised intersections;
- A few development opportunities were identified in the CAS system that would have made the crash data collection stage in this research more feasible, including:
    - Query of signalised intersection crashes in CAS should include give-way slip lanes control as well as signalised slip lanes control;
    - Develop a mechanism in CAS to identify left turn crashes for various left turn treatment types; and
    - Develop a method/system to identify crash location by approach and by lane configuration in an easy way without the need to review the TCR. For example, whether the crash occurred in through lane, right lane or left turn lane.

Opportunities for further research were identified as follows:

- Ideally, the sample size of the detailed crash analysis task should have increased from the current 276 approaches to include the whole signals network (1818 approaches). Moreover, it could have included more sites from other cities such as Christchurch and Wellington. This would have given more robust results for the crash analysis;
- Collect geometric data and design features of the left turn slip lanes (from the above samples) and subdividing them into more categories including:
    - Left turn slip lane types: high entry angle or low entry angle;
    - Island size and shape;

  - o Deceleration lane length; and
  - o Positions of pedestrian crossings at slip lanes.

  This is done to determine if any of these geometric features have any impact on the safety of the left turn slip lanes.

- ❖ Develop crash rate based on traffic/pedestrian volume exposure for the above samples. However, this would require considerable effort and resources to collect and analyse these data; and
- ❖ For the intersection performance, it is recommended to choose a four leg intersection, with a speed limit of 50k/hr, four left turn slip lanes, and operating at capacity. Then consideration should be given to modelling the effect of removing each of the left turn slip lanes individually and collectively on the intersection performance.

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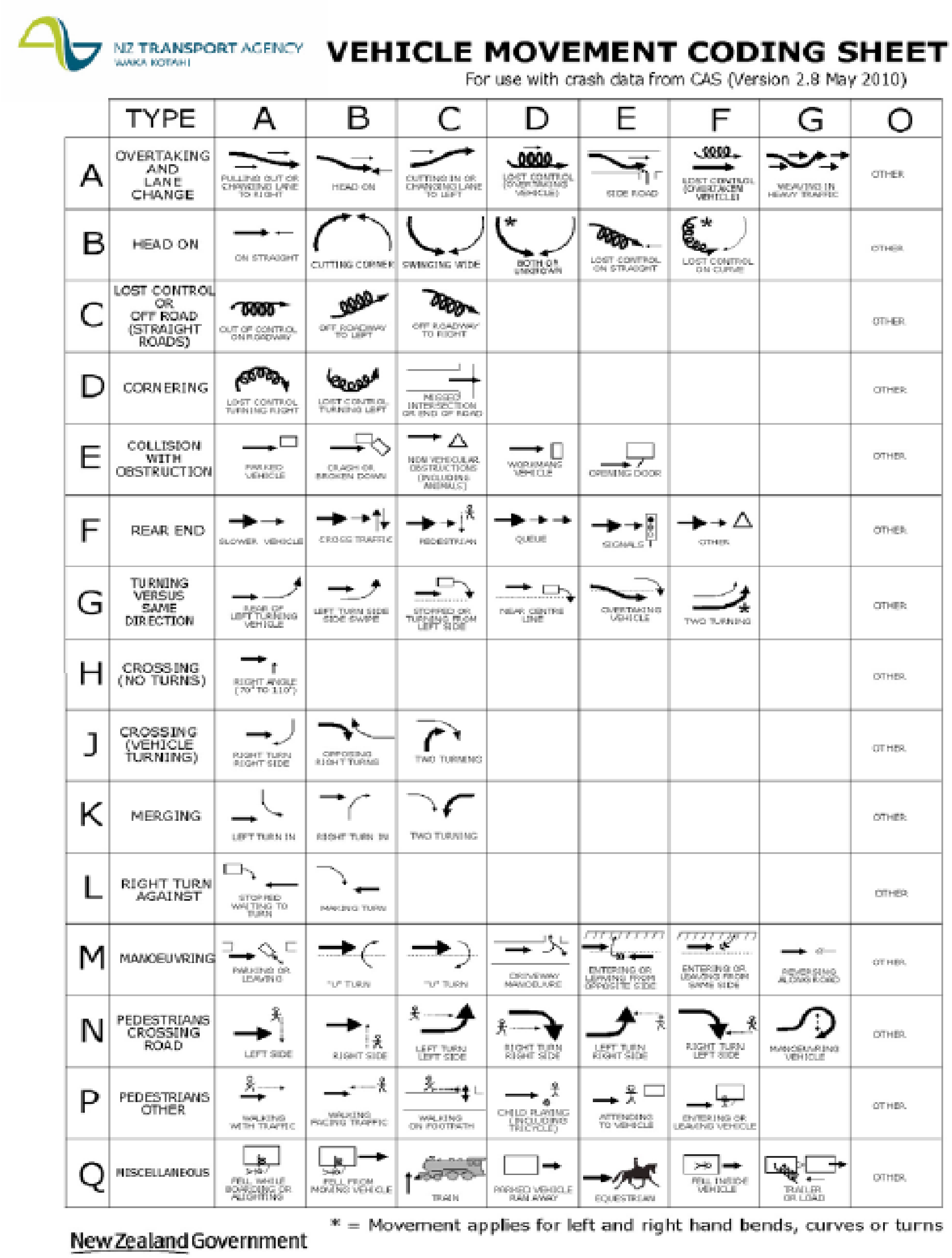
Appendix A CAS crash movement codes



NZ TRANSPORT AGENCY
WAKA KOTAHI

VEHICLE MOVEMENT CODING SHEET

For use with crash data from CAS (Version 2.8 May 2010)

	TYPE	A	B	C	D	E	F	G	O
A	OVERTAKING AND LANE CHANGE	PULLING OUT OR CHANGING LANE TO RIGHT	HEAD ON	CUTTING IN OR CHANGING LANE TO LEFT	LOST CONTROL (OVERTAKING VEHICLE)	SIDE ROAD	LOST CONTROL (OVERTAKEN VEHICLE)	WEAVING IN HEAVY TRAFFIC	OTHER
B	HEAD ON	ON STRAIGHT	CUTTING CORNER	SWINGING WIDE	BOTH OR UNKNOWN	LOST CONTROL ON STRAIGHT	LOST CONTROL ON CURVE		OTHER
C	LOST CONTROL OR OFF ROAD (STRAIGHT ROADS)	OUT OF CONTROL ON ROADWAY	OFF ROADWAY TO LEFT	OFF ROADWAY TO RIGHT					OTHER
D	CORNERING	LOST CONTROL TURNING RIGHT	LOST CONTROL TURNING LEFT	MISSED INTERSECTION OR END OF ROAD					OTHER
E	COLLISION WITH OBSTRUCTION	PARKED VEHICLE	CRASH OR BROKEN DOWN	NON VEHICULAR OBSTRUCTIONS (INCLUDING ANIMALS)	WORKMANS VEHICLE	OPENING DOOR			OTHER
F	REAR END	SLOWER VEHICLE	CROSS TRAFFIC	PEDESTRIAN	QUEUE	SIGNALS	OTHER		OTHER
G	TURNING VERSUS SAME DIRECTION	REAR OF LEFT TURNING VEHICLE	LEFT TURN SIDE SIDE SWIPE	STOPPED OR TURNING FROM LEFT SIDE	NEAR CENTRE LINE	OVERTAKING VEHICLE	TWO TURNING		OTHER
H	CROSSING (NO TURNS)	RIGHT ANGLE (70° TO 110°)							OTHER
J	CROSSING (VEHICLE TURNING)	RIGHT TURN RIGHT SIDE	OPPOSING RIGHT TURNS	TWO TURNING					OTHER
K	MERGING	LEFT TURN IN	RIGHT TURN IN	TWO TURNING					OTHER
L	RIGHT TURN AGAINST	STOPPED WAITING TO TURN	MAKING TURN						OTHER
M	MANOEUVRING	PARKING OR LEAVING	"U" TURN	"U" TURN	DRIVEWAY MANOEUVRE	ENTERING OR LEAVING FROM OPPOSITE SIDE	ENTERING OR LEAVING FROM SAME SIDE	REVERSING ALONG ROAD	OTHER
N	PEDESTRIANS CROSSING ROAD	LEFT SIDE	RIGHT SIDE	LEFT TURN LEFT SIDE	RIGHT TURN RIGHT SIDE	LEFT TURN RIGHT SIDE	RIGHT TURN LEFT SIDE	MANOEUVRING VEHICLE	OTHER
P	PEDESTRIANS OTHER	WALKING WITH TRAFFIC	WALKING FACING TRAFFIC	WALKING ON FOOTPATH	CHILD PLAYING (INCLUDING TRICYCLE)	ATTENDING TO VEHICLE	ENTERING OR LEAVING VEHICLE		OTHER
Q	MISCELLANEOUS	FELL WHILE BOARDING OR ALIGHTING	FELL FROM MOVING VEHICLE	TRAIN	PARKED VEHICLE RAN AWAY	EQUESTRIAN	FELL INSIDE VEHICLE	TRAILER OR LOAD	OTHER

New Zealand Government

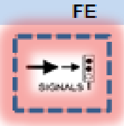
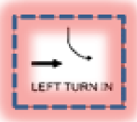
* = Movement applies for left and right hand bends, curves or turns

Source: NZ Transport Agency 2010

Appendix B Left turn crashes identification methodology

Appendix B.1 First attempt: the modified vehicle movement codes

The most likely of crash type and code for left turn slip lanes versus Conventional lanes

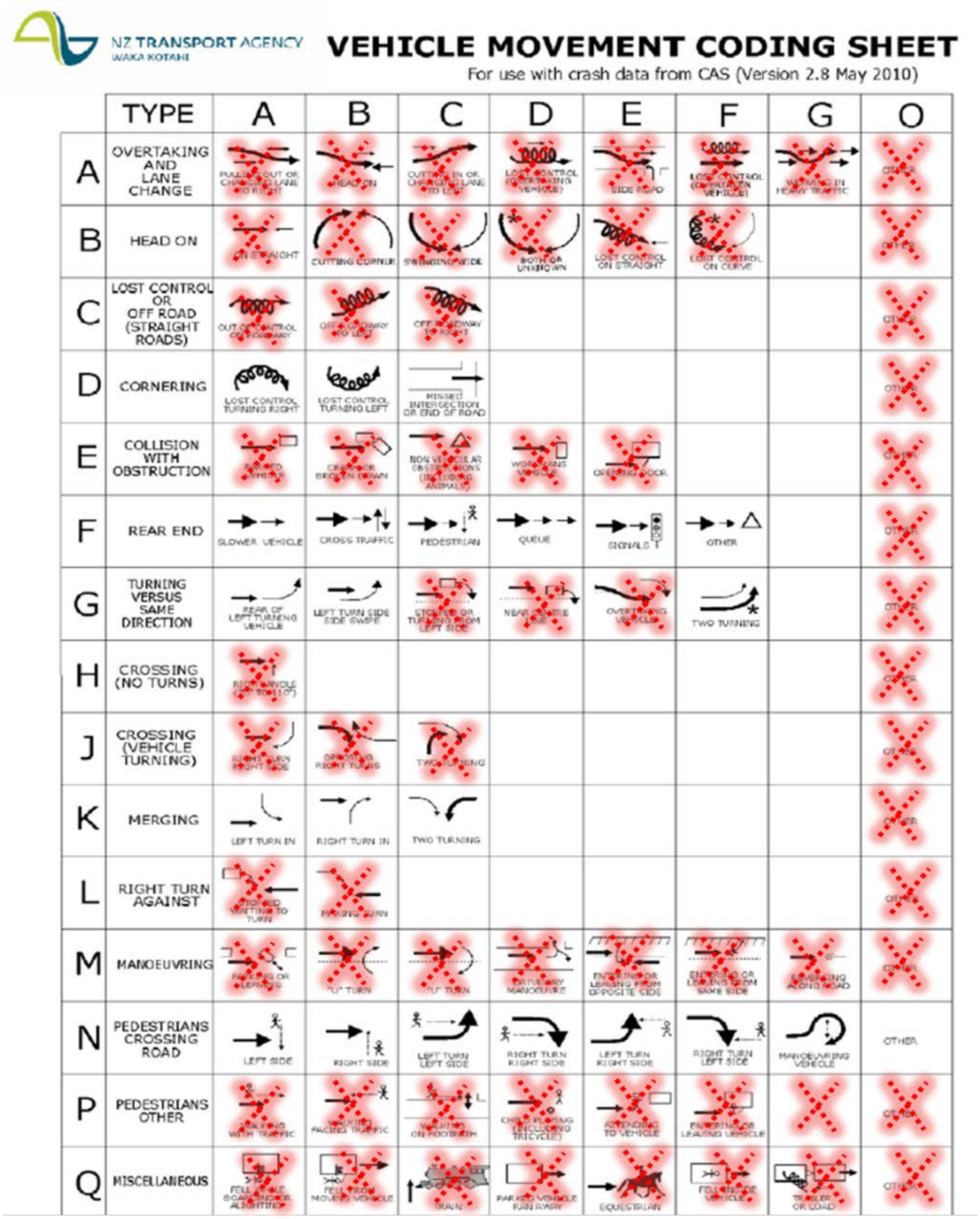
Crash Type and Code		Left Turn Type	
Crash Type	Crash Code	Slip Lanes	Conventional Lanes
Lost Control	CB		
		√	√
Rear End	FB		
		√	X
*Signalised Slip Lane only	FE		
		√	X
Turning V.S Same direction	GA		
		X	√
	GF		
		√	√
Merging	KA		
		√	X
Pedestrian Crossing Road	NC		
		√	√
	NE		
		√	√
LT Slip lanes	CB, FB. FE*, KA , GF, NC and NE		
LT Conventional lanes	CB,GA, GF,NC and NE		

Appendix B.2 Second attempt: the first version of the modified vehicle movement codes

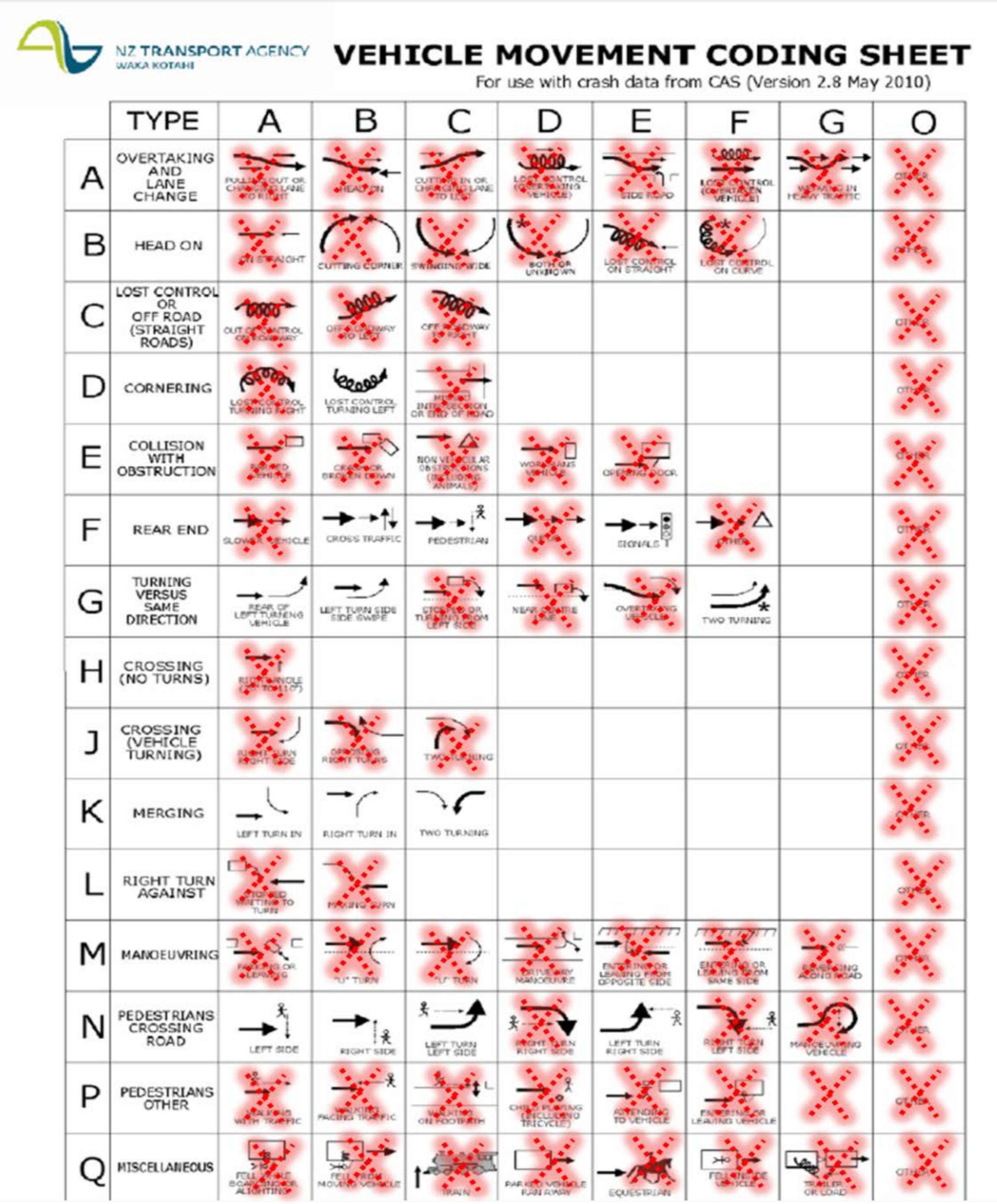
Left Turn Slip lanes Versus Conventional Left Turn Lanes at signalised Intersections			
Movement Code	**Slip Lane**	**Conventional Lane**	**Comment**
A types	Unlikely	Unlikely	Typically not at intersection
B types	Unlikely	Unlikely	Typically not at intersection
C types	Unlikely	Unlikely	Typically not at intersection
DA	Possibly	Possibly	Typically not at intersection
DB	Possibly	Possibly	Need to check each case
DC	Unlikely	Unlikely	Typically not related to left turn movement
DO	Unlikely	Unlikely	Need to check each case
E types	Unlikely	Unlikely	Not related to left turn movement
FA	Possibly	Possibly	Need to check each case
FB	Possibly	Possibly	Need to check each case
FC	Possibly	Possibly	Need to check each case
FD	Possibly	Possibly	Need to check each case
FE	Possibly	Possibly	Some slip lanes are signalised - check each case
FF	Possibly	Possibly	Need to check each case
FO	Unlikely	Unlikely	Need to check each case
GA	Possibly	Possibly	Need to check each case
GB	Possibly	Possibly	Need to check each case
GC	Unlikely	Unlikely	Typically not related to left turn movement
GD	Unlikely	Unlikely	Typically not related to left turn movement
GE	Unlikely	Unlikely	Typically not related to left turn movement
GF	Possibly	Possibly	Need to check each case
GO	Unlikely	Unlikely	Need to check each case
HA	Unlikely	Unlikely	Typically not related to left turn movement
HO	Unlikely	Unlikely	Typically not related to left turn movement
JA	Unlikely	Unlikely	Typically not related to left turn movement
JB	Unlikely	Unlikely	Typically not related to left turn movement
JC	Unlikely	Unlikely	Typically not related to left turn movement
JO	Unlikely	Unlikely	Typically not related to left turn movement
KA	Possibly	Possibly	Need to check each case
KB	Unlikely	Unlikely	Typically not related to left turn movement
KC	Possibly	Possibly	Need to check each case
KO	Unlikely	Unlikely	Need to check each case
LA	Unlikely	Unlikely	Typically not related to left turn movement
LB	Unlikely	Unlikely	Typically related to right turn filtering
LO	Unlikely	Unlikely	Related to right turn/stopping to make a turn

M type	Unlikely	Unlikely	Typically not at intersection
NA	Possibly	Possibly	Need to check each case
NB	Possibly	Possibly	Need to check each case
NC	Possibly	Possibly	Need to check each case
ND	Possibly	Possibly	Need to check each case
NE	Possibly	Possibly	Need to check each case
NF	Possibly	Possibly	Need to check each case
NG	Possibly	Possibly	Need to check each case
P types	Unlikely	Unlikely	Typically not at signalised intersection
Q types	Unlikely	Unlikely	Typically not related to left turn movement

Appendix B.3 Second attempt: the second version of the modified vehicle movement codes



Appendix B.4 Second attempt: the third version of the modified vehicle movement codes



Appendix C The detailed crash report of 230 crash records

Table below contains the detailed crash report for 230 crash records and intersection data used in the overall crash analysis task that presented in Chapter 4.

intersection No	intersection Name	CRASH ROAD	CRASH DIST	CRASH DIRN	INTSN	SIDE ROAD	CRASH ID	CRASH DATE	CRASH DOW	CRASH TIME	MVMT	LT Type	VEHICLES	CAUSES	OBJECTS ST	ROAD CURVE	ROAD WET	LIGHT	WEATHER	JUNC TYPE	TRAF CTRL	ROAD MARK	SPD LIM	CRASH FATA	CRASH SEV	CRASH MIN	PERS AGE1	PERS AGE2
2260	SEART / WAIPUNA RD	SOUTH-EASTERN HIGHWAY			I	WAIPUNA ROAD	201411624	16/03/2014	Sun	844	DB	ECL	CS2	131A 801	P	E	W	OF	L	T	T	R	50	0	0	1		
2182	WAIPUNA RD / CARBINE RD	CARBINE ROAD			I	SOUTH EASTERN HIGHWAY	201433794	11/03/2014	Tue	2115	DB	ECL	CW2	111A 131A 423A	P	M	D	DO	F	X	T	C	50	0	0	0		
2049	REMUERA RD / VICTORIA AVE / CLONBERN RD	VICTORIA AVENUE			I	REMUERA ROAD	201435237	13/03/2014	Thu	900	DB	ECL	BS1	129A 386A	S	R	D	O	F	T	T	C	50	0	0	0		
1934	SH16 / ACCESS ROAD (KUMEU)	16/19/4.551			I	ACCESS ROAD	201436482	21/05/2014	Wed	629	DB	ECL	CN2V	131A 133A		R	D	TO	F	T	T	C	50	0	0	0		
1807	SH1 (HIBISCUS COAST HWAY) / WEST HOE RD	HIBISCUS COAST HIGHWAY			I	WEST HOE ROAD	201441392	11/08/2014	Mon	1254	DB	ECL	TN1	129A 386A	KS	S	D	B	F	T	T	C	50	0	0	0		
1909	SH1 - NORTHCOTE RD INTERCHANGE	NORTHCOTE ROAD			I	NORTHCOTE ON NBD	201444397	3/10/2014	Fri	603	DB	ECL	CE1	131A 801	G	R	W	TO	L	X	T	C	100	0	0	0		
4902	SH20 OFFRAMP - RIMU RD - MAHUNGA DR	MAHUNGA DRIVE			I	RIMU OFF SBD	201437910	25/05/2014	Sun	1240	DB	FSL	CE1	111A 131A	T	E	D	BF	F	T	G	R	50	0	0	0		
4304	EAST TAMAKI RD - PRESTON RD	EAST TAMAKI ROAD			I	PRESTON ROAD	201414939	23/06/2014	Mon	215	DB	GSL	CN2	101A 111A 131A	I	E	D	DO	F	T	T	R	50	0	0	1		
4314	CHAPEL RD - ORMISTON RD	ORMISTON ROAD			I	CHAPEL ROAD	201415848	20/09/2014	Sat	1550	DB	GSL	CE1VC	131A 504A		R	W	O	L	X	T	P	60	0	0	1		
1904	SH18 - ALBANY HIGHWAY INTERCHANGE	ALBANY HIGHWAY ON EBD			I	ALBANY HIGHWAY	201430004	1/01/2014	Wed	1134	DB	GSL	CE2	131A 402A 403A		R	W	OF	L	T	G	C	50	0	0	0		
3015	GREAT NORTH RD / EDSEL ST / EDMONTON RD	EDMONTON ROAD			I	GREAT NORTH ROAD	201438594	25/06/2014	Wed	220	DB	GSL	CS1	103A 410A 801	IP	M	W	DO	L	X	T	R	50	0	0	0		
2921	STANLEY ST / ALTEN RD / NICHOLLS LN	ALTEN ROAD			I	16/0/1.21	201439239	27/06/2014	Fri	30	DB	GSL	CS1	111A 131A	KP	M	W	DO	L	X	T	C	50	0	0	0		
2190	GREAT SOUTH RD / CHURCH ST EAST	CHURCH ST			I	GREAT SOUTH ROAD	201443419	9/08/2014	Sat	2230	DB	GSL	CE1	386A 504A	I	S	D	DO	F	T	T	R	50	0	0	0		
4221	TI RAKAU DR - CHAPEL RD - DANNEMORA DR	CHAPEL ROAD			I	TI RAKAU DRIVE	201443479	13/09/2014	Sat	1640	DB	GSL	CN1	111A 131A	F	M	D	O	F	X	T	R	60	0	0	0		
4102	MCKENZIE RD - MILLER RD - CORONATION RD - WALMSLEY RD	CORONATION ROAD			I	WALMSLEY ROAD	201445536	12/09/2014	Fri	2150	DB	GSL	CS1	103A 111A 131A	IP	E	W	DO	L	X	T	R	50	0	0	0		
2252	LADIES MILE / MARUA RD	LADIES MILE			I	MARUA ROAD	201413822	9/05/2014	Fri	830	DB	SCL	CS1C	135A 806		M	W	B	F	T	T	C	50	0	0	1		
2378	GREAT NORTH RD / MCCRAE WAY	MCCRAE WAY			I	GREAT NORTH ROAD	201437141	27/05/2014	Tue	2300	DB	SCL	CS2	103A 131A	FJ	R	D	DO	F	T	S	C	50	0	0	0		

2077	WELLESLEY ST EAST / KITCHENER ST / MAYORAL DR	MAYORAL DRIVE			I	WELLESLEY ST EAST	201412385	8/02/2014	Sat	535	DB	SSL	CN2	103A 111A 131A 634A	J	E	W	DO	L	T	T	R	80	0	1	0		
2257	SEART / GREAT SOUTH RD	GREAT SOUTH ROAD			I	SOUTH-EASTERN HIGHWAY	201434844	28/03/2014	Fri	200	DB	SSL	CN1	103A 359A	P	R	D	DO	F	X	T	R	50	0	0	0		
4202	PAKURANGA RD - TI RAKAU DR	PAKURANGA ROAD			I	TI RAKAU DRIVE	201442640	17/07/2014	Thu	225	DB	SSL	CW1	130A 358A	S	R	D	DO	F	X	T	R	60	0	0	0		
4920	REDOUBT RD - SH1 IINTERCHANGE	REDOUBT ROAD			I	MANUKAU ON SBD	201443906	17/10/2014	Fri	1751	DB	SSL	CW1	111A 131A	G	R	W	OF	L	T	T	C	50	0	0	0		
2175	MANUKAU RD / PAH RD	MANUKAU ROAD			I	PAH ROAD	201444321	12/09/2014	Fri	1825	DB	SSL	CS1C	111A 131A 801		M	W	DO	L	T	T	R	50	0	0	0		
1305	LAKE RD - ESMONDE RD	ESMONDE ROAD			I	LAKE ROAD	201444893	12/11/2014	Wed	1822	DB	SSL	VN2	111A 131A		M	D	OF	F	T	T	C	50	0	0	0		
4206	TI RAKAU DR - REEVES RD - WAIPUNA EXPRESSWAY	PAKURANGA HIGHWAY			I	TI RAKAU DRIVE	201447733	28/10/2014	Tue	520	DB	SSL	VW1	111A 131A 801	S	M	W	DO	L	T	T	R	50	0	0	0		
1305	LAKE RD - ESMONDE RD	ESMONDE ROAD			I	LAKE ROAD	201447998	18/11/2014	Tue	2118	DB	SSL	VN2	111A 131A	IKS	E	D	DO	F	T	T	R	50	0	0	0		
4902	SH20 OFFRAMP - RIMU RD - MAHUNGA DR	MAHUNGA DRIVE			I	RIMU OFF SBD	201449952	24/12/2014	Wed	240	DB	SSL	CS2	111A 131A 514A	G	S	D	DO	F	T	G	R	50	0	0	0		
4919	GT STH RD - REDOUBT RD - MANUKAU STATION RD	GREAT SOUTH ROAD			I	MANUKAU STATION ROAD	201441171	8/08/2014	Fri	945	FB	FSL	CN1C	181A		E	D	BF	F	X	G	R	60	0	0	0		
4515	PUHINUI RD - CARRUTH RD - LAMBIE DR	PUHINUI ROAD			I	LAMBIE DRIVE	201411227	18/02/2014	Tue	937	FB	GSL	CW1C	181A 331A		E	D	O	F	R	G	R	50	0	0	1		
4521	CAVENDISH DR - LAMBIE DR	LAMBIE DRIVE			I	CAVENDISH DRIVE	201411694	21/03/2014	Fri	1300	FB	GSL	CN1C	181A		R	D	BF	F	X	T	C	50	0	0	1		
3015	GREAT NORTH RD / EDSEL ST / EDMONTON RD	EDMONTON ROAD			I	GREAT NORTH ROAD	201416077	13/10/2014	Mon	1550	FB	GSL	CS1C	181A 350A		M	D	B	F	M	G	C	50	0	0	1		
3015	GREAT NORTH RD / EDSEL ST / EDMONTON RD	EDMONTON ROAD			I	GREAT NORTH ROAD	201416083	10/10/2014	Fri	1307	FB	GSL	CS1P	181A 330A		E	D	B	F	X	T	R	50	0	0	1		
3908	SH16 - TE ATATU RD CITY BOUND RAMPS	TE ATATU ROAD			I	TE ATATU OFF EBD	201416737	20/10/2014	Mon	735	FB	GSL	CN2M	181A		M	D	B	F	T	G	N	50	0	0	1		
1902	SH1 - SH18 - CONSTELLATION DRIVE	UPPER HWY OFF SBD			I	CONSTELLATION DRIVE	201417111	12/09/2014	Fri	720	FB	GSL	CS1C	181A		R	W	O	L	T	G	P	50	0	0	1		
1904	SH18 - ALBANY HIGHWAY INTERCHANGE	ALBANY HIGHWAY			I	ALBANY HIGHWAY OFF EBD	201417459	6/10/2014	Mon	1558	FB	GSL	VN1C	181A		R	D	OF	F	X	T	C	50	0	0	1		
4612	ORMISTON RD - MURPHYS RD	ORMISTON ROAD			I	MURPHYS ROAD	201417654	2/12/2014	Tue	1820	FB	GSL	4E1C	181A 330A		R	D	B	F	X	G	P	50	0	0	1		
4207	PAKURANGA RD - BUCKLANDS BEACH RD - AVIEMORE DR	AVIEMORE DRIVE			I	PAKURANGA ROAD	201430598	22/01/2014	Wed	1130	FB	GSL	CN1C	181A		E	D	B	F	X	T	C	50	0	0	0		
4403	GT STH RD - BROWNS	GREAT SOUTH			I	ORAMS ROAD	20143069	10/02/2014	Mon	1720	FB	GSL	CS1C	181A		E	D	B	F	X	G	R	8	0	0	0		

	RD - ORAMS RD	ROAD					8																0					
1902	SH1 - SH18 - CONSTELLATION DRIVE	UPPER HWY OFF NBD			I	18/0/0.19	201432019	19/02/2014	Wed	2000	FB	GSL	CN1C	331A		R	D	O	F	T	G	N	50	0	0	0		
4426	ROSCOMMON RD - LANGLEY RD	HAUTU DRIVE			I	ROSCOMMON ROAD	201432365	3/03/2014	Mon	1711	FB	GSL	CE1C	181A 331A		R	D	TF	F	X	G	C	60	0	0	0		
3012	GREAT NORTH RD / HEPBURN RD	HEPBURN ROAD			I	GREAT NORTH ROAD	201432485	26/02/2014	Wed	835	FB	GSL	CS1C	331B 353B		E	D	BN	F	T	G	R	50	0	0	0		
4207	PAKURANGA RD - BUCKLANDS BEACH RD - AVIEMORE DR	BUCKLANDS BEACH ROAD			I	PAKURANGA ROAD	201432810	1/03/2014	Sat	1125	FB	GSL	TE2C	181A		E	D	B	F	X	T	N	60	0	0	0		
4919	GT STH RD - REDOUBT RD - MANUKAU STATION RD	REDOUBT ROAD			I	GREAT SOUTH ROAD	201433384	1/04/2014	Tue	1320	FB	GSL	CW1C	181A		E	D	BF	F	X	G	R	50	0	0	0		
3908	SH16 - TE ATATU RD CITY BOUND RAMPS	TE ATATU OFF EBD			I	TE ATATU ROAD	201434188	5/04/2014	Sat	900	FB	GSL	CN1C	331A		E	D	B	F	T	G	N	50	0	0	0		
4521	CAVENDISH DR - LAMBIE DR	CAVENDISH DRIVE			I	LAMBIE DRIVE	201434547	20/04/2014	Sun	1145	FB	GSL	CE1C	181A		R	W	OF	L	X	T	R	50	0	0	0		
3015	GREAT NORTH RD / EDSEL ST / EDMONTON RD	EDMONTON ROAD			I	GREAT NORTH ROAD	201435901	1/05/2014	Thu	1100	FB	GSL	CS1C	181A		E	D	BF	F	X	S	C	50	0	0	0		
4919	GT STH RD - REDOUBT RD - MANUKAU STATION RD	MANUKAU STATION ROAD			I	GREAT SOUTH ROAD	201436404	6/04/2014	Sun	2224	FB	GSL	CE1C	181A		R	D	DO	F	M	G	R	50	0	0	0		
4423	BROWNS RD - ROSCOMMON RD	BROWNS ROAD			I	ROSCOMMON ROAD	201437130	31/05/2014	Sat	1210	FB	GSL	TW1C	331A 353A		M	D	BF	F	X	T	N	50	0	0	0		
4501	GT STH RD - PUHINUI RD - REAGAN RD	GREAT SOUTH ROAD			I	REAGAN ROAD	201437202	10/05/2014	Sat	1329	FB	GSL	CS1C	181A		E	D	BF	F	X	G	C	50	0	0	0		
4207	PAKURANGA RD - BUCKLANDS BEACH RD - AVIEMORE DR	BUCKLANDS BEACH ROAD			I	PAKURANGA ROAD	201438456	17/06/2014	Tue	1625	FB	GSL	4S1V	181A 331A		E	D	B	F	X	T	C	50	0	0	0		
4423	BROWNS RD - ROSCOMMON RD	ROSCOMMON ROAD			I	BROWNS ROAD	201438651	1/07/2014	Tue	1440	FB	GSL	CW1C	181A		E	D	B	F	X	T	N	50	0	0	0		
4221	TI RAKAU DR - CHAPEL RD - DANNEMORA DR	TI RAKAU DRIVE			I	CHAPEL ROAD	201438800	4/07/2014	Fri	1530	FB	GSL	CE1C	387A 387B		E	D	BF	F	X	T	C	50	0	0	0		
4521	CAVENDISH DR - LAMBIE DR	LAMBIE DRIVE			I	CAVENDISH DRIVE	201438836	10/07/2014	Thu	1830	FB	GSL	CN1C	181A		R	W	DO	L	X	T	R	50	0	0	0		
2199	ELLERSLIE-PANMURE HWAY / LUNN AVE	LUNN AVENUE			I	ELLERSLIE-PANMURE HIGHWAY	201438942	14/06/2014	Sat	1615	FB	GSL	CS1C	181A 331A		R	D	BF	F	T	G	C	50	0	0	0		
4316	CHAPEL RD - ACCENT DR - STANCOMBE RD	ACCENT DRIVE			I	CHAPEL ROAD	201439262	21/06/2014	Sat	1119	FB	GSL	CE1C	181A 387A		E	D	O	F	X	T	C	60	0	0	0		
4406	GT STH RD - MAHIA RD	GREAT SOUTH ROAD	10	S		MAHIA ROAD	201440202	12/07/2014	Sat	1742	FB	GSL	CN1V	331A		R	W	DO	F	T	G		50	0	0	0		
4124	GREAT SOUTH RD / PORTAGE RD	PORTAGE ROAD			I	GREAT SOUTH ROAD	201440777	25/07/2014	Fri	1100	FB	GSL	CN14	181A		E	D	O	F	X	G	C	50	0	0	0		
4530	WIRI STATION RD - ROSCOMMON OIL TERMINAL	WIRI STATION ROAD			I	ROSCOMMON ROAD	201440898	26/06/2014	Thu	1440	FB	GSL	TW1C	181A		E	D	B	F	T	G	R	60	0	0	0		

4605	TE IRIRANGI DR - ORMISTON RD	ORMISTON ROAD			I	TE IRIRANGI DRIVE	201441021	15/07/2014	Tue	1745	FB	GSL	CE1C	181A		R	D	DO	F	X	G	R	50	0	0	0		
1924	SH1 - OTEHA VALLEY RD INTERCHANGE	OTEHA VALLEY ROAD			I	OTEHA VALLEY ON SBD	201442057	2/08/2014	Sat	1431	FB	GSL	CW1C	181A		R	D	OF	F	X	G	C	50	0	0	0		
4501	GT STH RD - PUHINUI RD - REAGAN RD	REAGAN ROAD			I	GREAT SOUTH ROAD	201442418	8/08/2014	Fri	2100	FB	GSL	CW1C	331A 353A		E	D	DO	F	Y	G	N	50	0	0	0		
4919	GT STH RD - REDOUBT RD - MANUKAU STATION RD	REDOUBT ROAD			I	GREAT SOUTH ROAD	201442431	5/08/2014	Tue	1530	FB	GSL	CW1C	387A		R	D	BF	F	X	G	R	50	0	0	0		
4501	GT STH RD - PUHINUI RD - REAGAN RD	REAGAN ROAD			I	GREAT SOUTH ROAD	201442532	9/08/2014	Sat	1930	FB	GSL	CW1C	331A 387A		E	D	DO	F	X	G	C	50	0	0	0		
4931	SH20 - CAVENDISH DR INTERCHANGE	PUHINUI OFF SBD			I	CAVENDISH DRIVE	201442698	1/10/2014	Wed	1630	FB	GSL	CE1C	181A		E	D	BF	F	X	G	N	50	0	0	0		
4521	CAVENDISH DR - LAMBIE DR	LAMBIE DRIVE			I	CAVENDISH DRIVE	201442744	7/09/2014	Sun	1700	FB	GSL	CS1C	181A		R	D	BF	F	X	T	R	50	0	0	0		
1931	SH18 - CARIBBEAN DR	CARIBBEAN DRIVE	5	S		SH 18	201443020	4/09/2014	Thu	1620	FB	GSL	CN1C	181A 331A 801		R	W	O	L	T	G	C	50	0	0	0		
4213	TI RAKAU DR - GOSSAMER DR - FREMANTLE PL	GOSSAMER DRIVE			I	TI RAKAU DRIVE	201443149	26/08/2014	Tue	842	FB	GSL	CS1C	181A		E	D	B	F	X	G	C	50	0	0	0		
4109	MASSEY RD - ROBERTSON RD	MASSEY ROAD			I	ROBERTSON ROAD	201443847	25/08/2014	Mon	815	FB	GSL	CN1C	181A		R	D	BF	F	X	G	C	50	0	0	0		
1102	WAIRAU RD - TRISTRAM AVE - HILLSIDE RD	HILLSIDE ROAD			I	WAIRAU ROAD	201444102	19/09/2014	Fri	1400	FB	GSL	CE1C	181A		R	D	OF	F	X	G	C	50	0	0	0		
4219	TI RAKAU DR - BOTANY RD - TE IRIRANGI DR	TE IRIRANGI DRIVE			I	TI RAKAU DRIVE	201444724	23/08/2014	Sat	900	FB	GSL	CW14	331A 387A		R	D	BF	F	X	T	C	60	0	0	0		
1406	GLENFIELD RD - BIRKENHEAD AVE - PUPUKE RD	PUPUKE ROAD			I	GLENFIELD ROAD	201446065	23/06/2014	Mon	645	FB	GSL	CS2C	181A 331A 801		E	W	O	L	T	G	R	50	0	0	0		
2911	SH16 / GREAT NORTH RD (WATERVIEW INTERCHANGE)	WATERVIEW OFF WBD			I	GREAT NORTH ROAD N	201446520	17/10/2014	Fri	2030	FB	GSL	CS1C	380A		E	W	DO	L	T	G	C	50	0	0	0		
2910	SH16 OFF-RAMP / NEWTON RD	NEWTON OFF EBD			I	NEWTON ROAD	201447148	31/10/2014	Fri	1130	FB	GSL	CE1C	181A 331A		E	D	O	F	T	G	C	50	0	0	0		
4521	CAVENDISH DR - LAMBIE DR	CAVENDISH DRIVE			I	LAMBIE DRIVE	201447218	13/11/2014	Thu	840	FB	GSL	CN1C	181A		E	W	O	L	X	T	L	50	0	0	0		
4512	WYLLIE RD - STATION RD	STATION ROAD			I	WYLLIE ROAD	201447373	13/11/2014	Thu	755	FB	GSL	CN1C	181A 330A		E	D	O	L	T	G	P	50	0	0	0		
1902	SH1 - SH18 - CONSTELLATION DRIVE	UPPER HWY OFF NBD			I	18/0/0.19	201447993	21/11/2014	Fri	705	FB	GSL	CN1V	181A 387A 191B		R	D	O	F	T	G	C	50	0	0	0		
4403	GT STH RD - BROWNS RD - ORAMS RD	BROWNS ROAD			I	GREAT SOUTH ROAD	201448037	6/11/2014	Thu	645	FB	GSL	CE1CC	181A 181B		E	D	B	F	X	T	C	80	0	0	0		
4102	MCKENZIE RD - MILLER RD - CORONATION RD - WALMSLEY RD	CORONATION ROAD			I	WALMSLEY ROAD	201448226	26/11/2014	Wed	1710	FB	GSL	CS1C	181A		E	D	B	F	X	T	L	50	0	0	0		
4418	ALFRISTON RD - PORCHESTER RD	PORCHESTER ROAD			I	ALFRISTON ROAD	201448358	15/12/2014	Mon	815	FB	GSL	CN1C	181A 330A		E	D	O	F	T	G	C	50	0	0	0		

1902	SH1 - SH18 - CONSTELLATION DRIVE	UPPER HWY OFF SBD			I	18/0/0.096	201449012	5/12/2014	Fri	1605	FB	GSL	CS1C	331A 353A		E	D	OF	F	R	G	N	50	0	0	0		
4523	CAVENDISH DRIVE - SHARKEY STREET	CAVENDISH DRIVE			I	SHARKEY ST	201449440	23/12/2014	Tue	1355	FB	GSL	CN2C	181A 387A		R	D	B	F	T	G	R	60	0	0	0		
4919	GT STH RD - REDOUBT RD - MANUKAU STATION RD	GREAT SOUTH ROAD			I	REDOUBT ROAD	201449649	9/12/2014	Tue	940	FB	GSL	CN1C	181A		R	D	O	L	X	G	R	80	0	0	0		
1810	SH1 (HIBISCUS COAST HWAY) / WHANGAPARAOA RD	SILVERDALE PARKWAY			I	HIBISCUS COAST HIGHWAY	201449859	19/12/2014	Fri	1320	FB	GSL	CE1C	181A		R	D	BF	F	X	G	C	50	0	0	0		
4108	WALMSLEY RD - MAHUNGA DR	MAHUNGA DRIVE			I	WALMSLEY ROAD	201449909	28/11/2014	Fri	1538	FB	GSL	CS1C	181A 331A		E	D	B	F	X	G	R	50	0	0	0		
4915	WIRI STATION RD - DRUCES RD	DRUCES ROAD			I	WIRI STATION ROAD	201449910	2/12/2014	Tue	800	FB	GSL	CN1C	181A 331A		E	D	TF	F	T	G	N	60	0	0	0		
4213	TI RAKAU DR - GOSSAMER DR - FREMANTLE PL	GOSSAMER DRIVE			I	TI RAKAU DRIVE	201450153	30/12/2014	Tue	1630	FB	GSL	CS1C	181A 331A		M	D	B	F	X	G	R	60	0	0	0		
2196	PENROSE RD / BARRACK RD	PENROSE ROAD			I	BARRACK ROAD	201416761	19/09/2014	Fri	1630	FB	SCL	CE1C	181A 387A		R	D	O	F	T	S	C	50	0	0	1		
4323	CHAPEL RD - DAWSON RD	DAWSON ROAD			I	CHAPEL ROAD	201449135	4/12/2014	Thu	1245	FB	SCL	CE1C	181A		R	D	OF	F	X	G	C	50	0	0	0		
4202	PAKURANGA RD - TI RAKAU DR	TI RAKAU DRIVE			I	PAKURANGA ROAD	201435315	14/04/2014	Mon	1800	FB	SSL	CN1C	181A		M	D	TO	F	T	G	C	70	0	0	0		
2316	COLLEGE RD / NGAHUE DR	NGAHUE DRIVE	5	W		COLLEGE ROAD	201416189	6/10/2014	Mon	1430	FB	ZSL	4E1C	331A 352A 845		E	D	O	F	T	G	C	50	0	0	1		
1407	GLENFIELD RD - KAIPATIKI RD	KAIPATIKI ROAD			I	GLENFIELD ROAD	201430869	22/02/2014	Sat	1900	FB	ZSL	CE1X	181A		E	D	BF	F	X	G	R	50	0	0	0		
2368	GREAT NORTH RD / TITIRANGI RD / RATA ST	TITIRANGI ROAD			I	GREAT NORTH ROAD	201431016	14/02/2014	Fri	1824	FB	ZSL	CW1C	181A 350A		E	W	O	F	T	G	C	50	0	0	0		
1110	TARGET RD - LINK DR	LINK DRIVE			I	TARGET ROAD	201433986	13/04/2014	Sun	1243	FB	ZSL	CW1C	181A 387A		M	D	B	F	T	G	C	50	0	0	0		
4302	EAST TAMAKI RD - BAIRDS RD	BAIRDS ROAD			I	EAST TAMAKI ROAD	201434702	16/04/2014	Wed	1215	FB	ZSL	CS1C	181A		E	D	BF	F	X	T	C	50	0	0	0		
4904	SH20 - MASSEY RD	MASSEY OFF NBD			I	MASSEY ROAD	201436418	28/03/2014	Fri	751	FB	ZSL	CN1C	181A		E	D	BF	F	T	G	N	50	0	0	0		
4610	TE IRIRANGI DR - BISHOP DUNN PL	TE IRIRANGI DRIVE			I	BISHOP DUNN PLACE	201437959	19/05/2014	Mon	840	FB	ZSL	CS1C	181A 402A		E	D	B	F	X	T	R	80	0	0	0		
1208	NORTHCOTE RD - SUNNYBRAE RD - AKORANGA DR	NORTHCOTE ROAD			I	AKORANGA DRIVE	201438125	13/05/2014	Tue	700	FB	ZSL	CS1C	181A		E	D	BF	F	X	T	C	50	0	0	0		
1506	ONEWA RD - BIRKENHEAD AVE - HIGHBURY BYPASS	BIRKENHEAD AVENUE			I	ONEWA ROAD	201439844	30/05/2014	Fri	1200	FB	ZSL	CN1T	181A 331A		R	D	B	F	X	T	R	50	0	0	0		
4202	PAKURANGA RD - TI RAKAU DR	TI RAKAU DRIVE			I	PAKURANGA ROAD	201440782	29/07/2014	Tue	1500	FB	ZSL	CN1C	181A		R	D	B	F	T	G	R	50	0	0	0		
1204	TAHAROTO RD - NORTHCOTE RD	TAHAROTO ROAD			I	NORTHCOTE ROAD	201442638	14/07/2014	Mon	1640	FB	ZSL	VN1V	353A 387A		R		BF	F	X	G	C	50	0	0	0		
4202	PAKURANGA RD - TI	TI RAKAU DRIVE			I	PAKURANGA	20144338	31/08/2014	Sun	1340	FB	ZSL	CN1V	181A		R	D	O	F	T	G	C	5	0	0	0		

	RAKAU DR					ROAD	5																0					
1110	TARGET RD - LINK DR	TARGET ROAD			I	LINK DRIVE	201443469	22/09/2014	Mon	1027	FB	ZSL	CW1C	331A 645A		E	W	O	F	T	G	R	50	0	0	0		
1208	NORTHCOTE RD - SUNNYBRAE RD - AKORANGA DR	NORTHCOTE ROAD			I	AKORANGA DRIVE	201443886	13/08/2014	Wed	945	FB	ZSL	CN1C	331A		R	D	BF	F	X	T	C	50	0	0	0		
1407	GLENFIELD RD - KAIPATIKI RD	KAIPATIKI ROAD			I	GLENFIELD ROAD	201447259	17/10/2014	Fri	2123	FB	ZSL	VS1C	181A 331A		E	D	DO	F	T	G	R	50	0	0	0		
4402	GT STH RD - ALFRISTON RD - WEYMOUTH RD	GREAT SOUTH ROAD			I	ALFRISTON ROAD	201450110	27/11/2014	Thu	2119	FB	ZSL	VS1C	103A 331A		E	D	DO	F	X	T	R	50	0	0	0		
1208	NORTHCOTE RD - SUNNYBRAE RD - AKORANGA DR	SUNNYBRAE ROAD			I	NORTHCOTE ROAD	201450522	23/12/2014	Tue	1430	FB	ZSL	CS1C	181A		E	D	O	F	X	G	R	50	0	0	0		
2153	NEW NORTH RD / MT ALBERT RD / CARRINGTON RD	NEW NORTH ROAD	5	W		MOUNT ALBERT ROAD	201441114	5/08/2014	Tue	1530	FC	SCL	TS1C	331A		R	D	BF	F	X	T	C	50	0	0	0		
3009	GREAT NORTH RD / WEST COAST RD	GREAT NORTH ROAD			I	WEST COAST ROAD	201431556	8/01/2014	Wed	1230	FC	ZSL	CW1C	181A 331A		M	D	O	F	T	T	X	50	0	0	0		
2914	GREAT NORTH RD / ST LUKES RD EXTN.	ST LUKES ROAD	10	S		GREAT NORTH ROAD	201431625	17/01/2014	Fri	2342	FC	ZSL	CN1C	181A		E	D	DO	F	T	G	X	50	0	0	0		
2054	QUAY ST / COMMERCE ST / GORE ST	QUAY ST			I	COMMERCE ST	201430497	6/02/2014	Thu	1705	FE	ECL	CW1C	181A		R	D	O	F	T	T	C	50	0	0	0		
3011	GREAT NORTH RD / GLENVIEW RD / SABULITE RD	GLENVIEW ROAD			I	GREAT NORTH ROAD	201432022	14/02/2014	Fri	2230	FE	ECL	CN1C	420B		R	D	DO	F	X	T	C	50	0	0	0		
2354	CLARK ST / WARD ST	WARD ST			I	CLARK ST	201432269	29/01/2014	Wed	1812	FE	ECL	CS1C	181A 331A		R	D	B	F	T	T	C	50	0	0	0		
2342	ABBOTTS WAY / LUNN AVE / NGAHUE DR	ABBOTTS WAY	10	W		NGAHUE DRIVE	201434356	11/04/2014	Fri	1730	FE	ECL	CE1C	331A		R	D	TO	F	X	T	C	50	0	0	0		
2130	VICTORIA ST EAST / KITCHENER ST / BOWEN AVE	VICTORIA ST EAST			I	KITCHENER ST	201436376	26/03/2014	Wed	1600	FE	ECL	4E1C	181A		M	D	BF	F	T	T	C	50	0	0	0		
2197	CARBINE RD / PANAMA RD	CARBINE ROAD			I	PANAMA ROAD	201436865	26/05/2014	Mon	750	FE	ECL	CS1M	130A 350A		E	W	O	L	T	T	C	40	0	0	0		
2259	SEART / CARBINE RD	SOUTH-EASTERN HIGHWAY			I	CARBINE ROAD	201440927	8/08/2014	Fri	1400	FE	ECL	CW1C	181A		R	D	BF	F	X	T	R	50	0	0	0		
4404	KERRS RD - DALGETY DR	DALGETY DRIVE			I	KERRS ROAD	201441022	12/07/2014	Sat	1635	FE	ECL	CN1C	181A 387A		R	W	OF	L	T	T	C	60	0	0	0		
2008	QUEEN ST / UPPER QUEEN ST / KARANGAHAPE RD	KARANGAHAPE ROAD			I	QUEEN ST	201442626	10/07/2014	Thu	1155	FE	ECL	CE1C	181A		R	W	OF	F	X	T	C	50	0	0	0		
1409	GLENFIELD RD - ALBANY HWY - SUNSET RD - GLENDHU RD	ALBANY HIGHWAY			I	SUNSET ROAD	201443153	20/08/2014	Wed	1123	FE	ECL	VS1C	181A 331A		R	W	O	F	X	T	C	50	0	0	0		
6004	GT STH RD / O'SHANNESSEY ST	OSHANNESSEY ST			I	GREAT SOUTH ROAD	201448361	10/12/2014	Wed	1247	FE	ECL	CW1C	181A 330A 191B		R	D	B	F	T	T	C	50	0	0	0		
6015	BROADWAY / O'SHANNESSEY ST	OSHANNESSEY ST			I	BROADWAY	201449171	10/12/2014	Wed	1406	FE	ECL	CS1C	181A		R	D	BF	F	T	T	C	50	0	0	0		

4307	EAST TAMAKI RD - BIRMINGHAM RD	EAST TAMAKI ROAD			I	BIRMINGHAM ROAD	201449648	9/12/2014	Tue	1113	FE	ECL	CW2C	181A		R	D	O	F	T	T	C	50	0	0	0		
2093	QUEEN ST / MAYORAL DR	QUEEN ST			I	MAYORAL DRIVE	201450528	27/12/2014	Sat	1632	FE	ECL	CN1C	181A 427A		R	D	B	F	X	T	C	50	0	0	0		
4207	PAKURANGA RD - BUCKLANDS BEACH RD - AVIEMORE DR	BUCKLANDS BEACH ROAD			I	PAKURANGA ROAD	201436829	4/05/2014	Sun	1315	FE	GSL	CS1C	181A 331A		E	D	O	F	X	T	C	50	0	0	0		
4403	GT STH RD - BROWNS RD - ORAMS RD	GREAT SOUTH ROAD			I	ORAMS ROAD	201442714	5/09/2014	Fri	1730	FE	GSL	CS1C	181A		R	W	DO	L	X	T	C	50	0	0	0		
4314	CHAPEL RD - ORMISTON RD	CHAPEL ROAD			I	ORMISTON ROAD	201444731	30/08/2014	Sat	2210	FE	GSL	CN1V	181A		R	D	DO	F	X	T	C	50	0	0	0		
1611	GREVILLE RD - HUGH GREEN DR	HUGH GREEN DRIVE			I	GREVILLE ROAD	201444750	23/08/2014	Sat	1000	FE	GSL	CW1C	181A		E	D	BF	F	X	T	R	50	0	0	0		
4112	KIRKBRIDE RD - ASCOT RD	KIRKBRIDE ROAD			I	ASCOT ROAD	201446862	13/11/2014	Thu	2227	FE	GSL	CE1CX	103A 112A 431A 103B 112B 431B		R	W	DO	F	T	T	C	80	0	0	0		
3022	LINCOLN RD - TE PAI PL - POMARIA RD	LINCOLN ROAD			I	POMARIA ROAD	201415321	30/08/2014	Sat	850	FE	SCL	CN1C	181A 334A		R	D	B	F	X	T	C	50	0	0	1		
1307	ESMONDE RD - BURNS AVE	BURNS AVENUE			I	ESMONDE ROAD	201415469	8/08/2014	Fri	1318	FE	SCL	CS1CC	181A 181B		R	D	O	F	T	T	C	50	0	0	1		
4125	GREAT SOUTH RD / HUIA RD / ALBION ST	GREAT SOUTH ROAD			I	HUIA ROAD	201415673	25/09/2014	Thu	1025	FE	SCL	BN1C	181A 331A		R	D	B	F	X	T	C	70	0	0	1		
2179	GREAT SOUTH RD / ROCKFIELD RD	GREAT SOUTH ROAD			I	ROCKFIELD ROAD	201416918	5/09/2014	Fri	1544	FE	SCL	4N1C	181A		R	W	OF	L	T	T	R	50	0	0	1		
1205	TAHAROTO RD - SHAKESPEARE RD - WAIRAU RD	WAIRAU ROAD			I	SHAKESPEARE ROAD	201430384	15/01/2014	Wed	1642	FE	SCL	CN1C	181A 351A		R	D	B	F	M	T	C	80	0	0	0		
3048	LINCOLN RD - NORVAL RD	LINCOLN ROAD			I	NORVAL ROAD	201430756	16/01/2014	Thu	1630	FE	SCL	CS1C	181A		R	D	OF	F	T	T	C	50	0	0	0		
2004	SYMONDS ST / KARANGAHAPE RD / GRAFTON BRIDGE	GRAFTON BRIDGE			I	SYMONDS ST	201430787	10/01/2014	Fri	1930	FE	SCL	CW1C	181A		R	D	BF	F	X	T	C	50	0	0	0		
2147	HILLSBOROUGH RD / HERD RD / CARR RD	HILLSBOROUGH ROAD			I	CARR ROAD	201430923	28/01/2014	Tue	1935	FE	SCL	CN1C	181A		R	D	BF	F	T	T	C	50	0	0	0		
4300	EAST TAMAKI RD - FERGUSON RD	EAST TAMAKI ROAD			I	FERGUSON ROAD	201432054	7/02/2014	Fri	1815	FE	SCL	CW1C	181A 331A 387A 801		R	W	TO	L	X	T	R	50	0	0	0		
2248	REMUERA RD / MIDDLETON RD	REMUERA ROAD	10	N		MIDDLETON ROAD	201433146	31/03/2014	Mon	1705	FE	SCL	VW1S	181A		R	D	B	F	T	T	C	50	0	0	0		
2146	DOMINION RD / MT ALBERT RD	DOMINION ROAD			I	MOUNT ALBERT ROAD	201433763	10/02/2014	Mon	1100	FE	SCL	CN1C	181A		R	D	B	F	X	T	C	50	0	0	0		
2159	MT SMART RD / ONEHUNGA MALL	ONEHUNGA MALL			I	MOUNT SMART ROAD	201435256	24/03/2014	Mon	1019	FE	SCL	CN1C	181A		R	D	B	F	X	T	P	50	0	0	0		
2153	NEW NORTH RD / MT ALBERT RD /	MOUNT ALBERT ROAD			I	NEW NORTH ROAD	201435345	9/04/2014	Wed	1510	FE	SCL	CW1C	181A		R	D	B	F	X	T	C	50	0	0	0		

	CARRINGTON RD																											
2145	MT ALBERT RD / MT EDEN RD / WARREN AVE	MOUNT ALBERT ROAD			I	WARREN AVENUE	201437165	6/05/2014	Tue	1932	FE	SCL	CN2C	181A		R	W	OF	L	X	T	C	50	0	0	0		
2141	DOMINION RD / VIEW RD / GEORGE ST	DOMINION ROAD			I	VIEW ROAD	201437292	4/06/2014	Wed	1715	FE	SCL	4S1X	181A		R	D	BF	F	T	T	L	50	0	0	0		
3020	LINCOLN RD - SEL PEACOCK DRV	SEL PEACOCK DRIVE			I	LINCOLN ROAD	201437374	24/05/2014	Sat	1919	FE	SCL	4W1C	103A 331A		E	D	DO	F	T	T	R	50	0	0	0		
3901	SH16 - TE ATATU RD	TE ATATU ROAD			I	TE ATATU ON WBD	201438713	8/07/2014	Tue	815	FE	SCL	CN1C	181A		R	D	OF	F	T	T	C	50	0	0	0		
2147	HILLSBOROUGH RD / HERD RD / CARR RD	HILLSBOROUGH ROAD			I	CARR ROAD	201438938	25/06/2014	Wed	1720	FE	SCL	CN1C	331A 387A		R	W	DO	L	T	T	C	50	0	0	0		
2087	GREEN LANE EAST / ASCOT AVE	ASCOT AVENUE			I	GREEN LANE EAST	201439667	3/07/2014	Thu	1115	FE	SCL	CE1C	181A		E	W	O	F	T	T	C	50	0	0	0		
4406	GT STH RD - MAHIA RD	MAHIA ROAD			I	GREAT SOUTH ROAD	201439949	20/06/2014	Fri	1515	FE	SCL	CE1C	181A		R	D	OF	F	T	T	C	50	0	0	0		
4406	GT STH RD - MAHIA RD	MAHIA ROAD			I	GREAT SOUTH ROAD	201440185	21/07/2014	Mon	910	FE	SCL	CE1C	330A 181		R	D	B	F	T	T	C	50	0	0	0		
2002	SYMONDS ST / KHYBER PASS RD / NEWTON RD	NEWTON ROAD			I	SYMONDS ST	201440902	4/08/2014	Mon	1300	FE	SCL	CE1C	350A 387A		R	D	BF	F	X	T	C	50	0	0	0		
1204	TAHAROTO RD - NORTHCOTE RD	TAHAROTO ROAD			I	NORTHCOTE ROAD	201442122	21/08/2014	Thu	1741	FE	SCL	CS1C	181A		R	D	O	F	X	T	C	50	0	0	0		
2335	MAY RD / STODDARD RD / DENBIGH AVE	STODDARD ROAD			I	MAY ROAD	201442695	1/10/2014	Wed	2046	FE	SCL	CN1C	181A		R	D	OF	F	X	T	C	50	0	0	0		
4303	EAST TAMAKI RD - SPRINGS RD	EAST TAMAKI ROAD			I	SPRINGS ROAD	201410090	17/02/2014	Mon	1900	FE	SSL	CW1V	103A 331A		M	D	TF	F	T	T	C	60	0	0	1		
3901	SH16 - TE ATATU RD	TE ATATU OFF WBD	10	E		TE ATATU ROAD	201431236	3/03/2014	Mon	715	FE	SSL	CW1C	181A		E	W	OF	F	T	T	N	50	0	0	0		
2368	GREAT NORTH RD / TITIRANGI RD / RATA ST	GREAT NORTH ROAD			I	TITIRANGI ROAD	201434267	18/02/2014	Tue	930	FE	SSL	CE1C	331A		R	D	B	F	X	T	C	50	0	0	0		
2918	SH20 ON & OFF-RAMPS / NEILSON ST / GLOUCESTER PARK RD	NEILSON ST	10	W		ONEHUNGA ON SBD	201436336	4/03/2014	Tue	759	FE	SSL	TW1CC	181A		E	D	BF	F	X	T	C	50	0	0	0		
2038	VICTORIA ST WEST / WELLESLEY ST / HALSEY ST	WELLESLEY ST WEST			I	VICTORIA ST WEST	201439606	20/06/2014	Fri	2320	FE	SSL	CN1C	181A 387A		R	D	DO	F	T	T	C	50	0	0	0		
4920	REDOUBT RD - SH1 IINTERCHANGE	REDOUBT ROAD			I	MANUKAU ON SBD	201447368	14/11/2014	Fri	1423	FE	SSL	CW1V	103A 112A		R	D	B	F	X	T	R	50	0	0	0		
4903	KIRKBRIDE RD - GEORGE BOLT MEM - SH 20A	20A/0/2.107			I	KIRKBRIDE ROAD	201449101	25/11/2014	Tue	620	FE	SSL	TS1C	331A 350A		R	D	OF	F	X	T	R	100	0	0	0		
4202	PAKURANGA RD - TI RAKAU DR	TI RAKAU DRIVE	5	S		PAKURANGA ROAD	201431550	25/01/2014	Sat	1805	FE	ZSL	CN1C	331A		R	D	B	F	T	T	C	50	0	0	0		
2008	QUEEN ST / UPPER QUEEN ST / KARANGAHAPE RD	QUEEN ST			I	KARANGAHAPE ROAD	201432119	28/02/2014	Fri	1500	FE	ZSL	CS1C	181A 330A		M	D	O	F	X	T	C	50	0	0	0		
2913	SH16 WEST BOUND ON & OFF-RAMPS / ST LUKES RD EXTN.	ST LUKES OFF WBD			I	ST LUKES ROAD	201441119	17/07/2014	Thu	1100	FE	ZSL	MW1C	181A		R	D	BF	F	T	T	N	50	0	0	0		

4229	TI RAKAU DR - TRUGOOD DR	TRUGOOD DRIVES			I	TI RAKAU DRIVE	201413931	10/05/2014	Sat	1338	GA	GSL	CN1C	181A		R	D	B	F	T	G	C	50	0	0	1		
2916	SH20 / HILLSBOROUGH RD INTERCHANGE	HILLSBOROUGH OFF EBD	5	W		HILLSBOROUGH ROAD	201435601	19/04/2014	Sat	1225	GA	GSL	CE1C	181A 331A		E	D	B	F	X	G	R	50	0	0	0		
2254	BEACH RD / TANGIHUA ST	BEACH ROAD			I	TANGIHUA ST	201437865	7/05/2014	Wed	2045	GA	GSL	CS1C	181A 103B 191B		E	W	DO	F	T	T	C	50	0	0	0		
4612	ORMISTON RD - MURPHYS RD	ORMISTON ROAD			I	MURPHYS ROAD	201438579	15/06/2014	Sun	1115	GA	GSL	CE1C	181A		R	D	O	F	X	G	R	50	0	0	0		
2013	GREEN LANE WEST / MANUKAU RD	MANUKAU ROAD			I	GREEN LANE WEST	201410179	22/01/2014	Wed	1510	GA	SCL	CS1C	181A 331A		R	D	B	F	X	T	C	50	0	0	1		
2155	MT ALBERT RD / SANDRINGHAM RD	MOUNT ALBERT ROAD	10	E		SANDRINGHAM ROAD	201431303	2/02/2014	Sun	1015	GA	SCL	CE14	181A 927		R		F		D	T	C	50	0	0	0		
4136	PRINCES ST / FRANK GREY PL (OTAHUHU)	FRANK GREY PLACE			I	PRINCES ST EAST	201433610	22/03/2014	Sat	2222	GA	SCL	CS14	103A 181A 331A 402A		R	D	DO	F	X	T	C	50	0	0	0		
2005	KARANGAHAPE RD / PITT ST / MERCURY LN	KARANGAHAPE ROAD			I	MERCURY LANE	201435482	1/05/2014	Thu	800	GA	SCL	BW1C	181A 132B 194B		R	D	B	F	X	T	C	50	0	0	0		
2366	GREAT NORTH RD / PORTAGE RD	GREAT NORTH ROAD			I	PORTAGE ROAD	201433940	1/04/2014	Tue	1420	GA	ZSL	CW1C	181A 387A 142B		R	D	B	F	T	G	C	50	0	0	0		
4302	EAST TAMAKI RD - BAIRDS RD	BAIRDS ROAD			I	EAST TAMAKI ROAD	201441962	15/08/2014	Fri	830	GA	ZSL	CN1C	181A 331A 801		E	W	O	L	X	T	C	50	0	0	0		
4536	MANUKAU STATION - WIRI STATION RD (was 4928)	MANUKAU STATION ROAD			I	WIRI STATION ROAD	201437278	6/06/2014	Fri	706	GF	ECL	TW1C	381A 671A		R	W	BF	F	T	T	C	60	0	0	0		
4503	GT STH RD - EAST TAMAKI RD	GREAT SOUTH ROAD			I	EAST TAMAKI ROAD	201437513	23/05/2014	Fri	940	GF	ECL	VW2C	129B 503B		E	D	B	F	T	T	N	50	0	0	0		
2051	PONSONBY RD / RICHMOND RD / PICTON ST	PONSONBY ROAD			I	RICHMOND ROAD	201440674	17/07/2014	Thu	1024	GF	ECL	TE2C	172A 370A		R	D	B	F	X	T	C	40	0	0	0		
2259	SEART / CARBINE RD	CARBINE ROAD			I	SOUTH-EASTERN HIGHWAY	201442607	6/07/2014	Sun	1614	GF	ECL	CW2C	381A 381B		R	D	BF	F	X	T	C	50	0	0	0		
1617	ALBANY HWAY - OAKWAY DRIVE	ALBANY HIGHWAY			I	OAKWAY DRIVE	201444335	26/09/2014	Fri	1222	GF	ECL	CN1C	172A 355A 372A 404A		E	D	O	F	T	T	C	50	0	0	0		
2001	ALBERT ST / CUSTOMS ST WEST / FANSHAWE ST	LOWER ALBERT ST			I	CUSTOMS ST WEST	201450208	23/12/2014	Tue	1730	GF	ECL	BS1C	671A		R	D	DO	F	X	T	C	50	0	0	0		
3019	GREAT NORTH RD / BUSCOMB AVE / SWANSON RD / LINCOLN RD	BUSCOMB AVENUE			I	GREAT NORTH ROAD	201440488	1/08/2014	Fri	914	GF	GSL	4W1C	330B 387B		S	W	O	M	M	T	R	50	0	0	0		
4304	EAST TAMAKI RD - PRESTON RD	EAST TAMAKI ROAD			I	PRESTON ROAD	201447605	20/11/2014	Thu	1220	GF	GSL	CW1C	372A		R	D	BF	F	T	G	P	50	0	0	0		

2041	VICTORIA ST WEST / ALBERT ST	VICTORIA ST WEST			I	ALBERT ST	201436218	12/04/2014	Sat	945	GF	SCL	BE1X	372A		M	D	B	F	X	T	C	50	0	0	0		
4403	GT STH RD - BROWNS RD - ORAMS RD	GREAT SOUTH ROAD			I	BROWNS ROAD	201437177	10/01/2014	Fri	820	GF	SCL	VN1V	172A		R	D	OF	F	X	T	C	50	0	0	0		
2019	QUEEN ST / WELLESLEY ST	WELLESLEY ST WEST			I	QUEEN ST	201444977	1/10/2014	Wed	1200	GF	SCL	4N1C	172A 381A		R	D	B	F	X	T	C	50	0	0	0		
2088	ST HELIERS BAY RD / ASHBY AVE / LONG DR	ST HELIERS BAY ROAD			I	LONG DRIVE	201449867	3/12/2014	Wed	1615	GF	SCL	BE1C	381A 671A		R	D	BF	F	X	T	C	50	0	0	0		
2919	SH1 ON-RAMP / SEART	SOUTH-EASTERN HIGHWAY			I	SEART ON NBD	201438941	23/06/2014	Mon	938	GF	SSL	TE1C	381A 671A		R	D	BF	F	M	T	C	50	0	0	0		
2064	KOHIMARAMA RD / ST JOHNS RD / ST HELIERS BAY RD	ST HELIERS BAY ROAD			I	ST JOHNS ROAD	201433562	11/04/2014	Fri	1800	GF	ZSL	CW1C	381A		E		F		T	G	C	50	0	0	0		
2013	GREEN LANE WEST / MANUKAU RD	MANUKAU ROAD			I	GREEN LANE WEST	201441187	27/06/2014	Fri	1700	GF	ZSL	MW2C	381A		R	D	BF	F	X	T	C	60	0	0	0		
2905	PARNELL RISE / BEACH RD / STANLEY ST / THE STRAND	16/0/0.982			I	PARNELL RISE	201400006	7/01/2014	Tue	1415	KA	ECL	TS1S	322B		S	D	B	F	X	T	C	50	1	0	0		37
1611	GREVILLE RD - HUGH GREEN DR	GREVILLE ROAD			I	HUGH GREEN DRIVE	201411080	15/01/2014	Wed	1935	KA	GSL	CS1C	302B 372B 402B		E	D	O	F	X	G	R	50	0	0	1		
2009	CUSTOMS ST E. / BEACH RD / BRITOMART PL / ANZAC AV / FORT ST	BEACH ROAD			I	BRITOMART PLACE W	201413834	27/05/2014	Tue	900	KA	GSL	SE1C	302B 375B 404B		R	D	B	F	T	G	N	50	0	0	1		47
4213	TI RAKAU DR - GOSSAMER DR - FREMANTLE PL	TI RAKAU DRIVE			I	GOSSAMER DRIVE	201416014	20/09/2014	Sat	720	KA	GSL	SS1CS	302B 375B		R	W	O	F	T	G	C	50	0	0	1		
4607	TE IRIRANGI DR - SMALES RD	TE IRIRANGI DRIVE			I	SMALES ROAD	201416221	5/08/2014	Tue	1710	KA	GSL	CS1V	302B		R	D	O	F	X	G	C	80	0	0	1		
4501	GT STH RD - PUHINUI RD - REAGAN RD	REAGAN ROAD			I	GREAT SOUTH ROAD	201417273	2/11/2014	Sun	1759	KA	GSL	SE1C	302B		R	D	TF	F	X	G	R	50	0	0	1		20
1612	CONSTELLATION DR - PARKWAY DR - HOME PLACE	CONSTELLATION DRIVE			I	PARKWAY DRIVE W	201430967	19/02/2014	Wed	1615	KA	GSL	BS24	302B 375B		R	D	OF	F	T	G	C	50	0	0	0		
1107	WAIRAU RD - PORANA RD	WAIRAU ROAD			I	PORANA ROAD	201431269	3/03/2014	Mon	1900	KA	GSL	CN1C	302B 375B		E	D	BF	F	X	G	R	50	0	0	0		
4607	TE IRIRANGI DR - SMALES RD	TE IRIRANGI DRIVE			I	SMALES ROAD	201433303	4/04/2014	Fri	1525	KA	GSL	CN1CC	302B 382B		R	D	OF	F	X	G	R	80	0	0	0		
4532	GT STH RD - LAKEWOOD CT	LAKEWOOD COURT			I	GREAT SOUTH ROAD	201435178	8/04/2014	Tue	615	KA	GSL	CS1C	302B 375B		R	D	B	F	T	G	C	50	0	0	0		
4915	WIRI STATION RD - DRUCES RD	LAMBIE DRIVE			I	WIRI STATION ROAD	201436559	7/05/2014	Wed	1900	KA	GSL	CS1T	130A 181A 331A		M	D	DO	F	X	T	C	50	0	0	0		
2244	KEPA RD / PATTESON AVE	KEPA ROAD			I	PATTESON AVENUE	201439665	1/07/2014	Tue	630	KA	GSL	CW1C	302B		E	W	TO	F	X	G	R	50	0	0	0		
3024	LINCOLN RD - TRIANGLE RD - CENTRAL PARK DRV	CENTRAL PARK DRIVE			I	LINCOLN ROAD	201442313	8/08/2014	Fri	939	KA	GSL	VE1C	322B 375B		E	D	B	F	X	T	C	50	0	0	0		
2172	CHURCH ST / O'RORKE	CHURCH ST			I	HUGO JOHNSTON	20144288	8/09/2014	Mon	712	KA	GSL	TW1C	322B		R	D	O	F	X	T	R	5	0	0	0		

	RD / HUGO JOHNSTON DR					DRIVE	3							351B									0					
3018	GREAT NORTH RD / HENDERSON VALLEY RD / ALDERMAN DRV	ALDERMAN DRIVE			I	GREAT NORTH ROAD	201443084	5/09/2014	Fri	620	KA	GSL	CN1V	184B 303B 801		R	W	TO	L	X	T	C	50	0	0	0		
1931	SH18 - CARIBBEAN DR	18/0/0.575			I	CARIBBEAN DRIVE	201446570	15/10/2014	Wed	1435	KA	GSL	CW1V	322B		R	D	B	F	T	T	C	80	0	0	0		
1611	GREVILLE RD - HUGH GREEN DR	GREVILLE ROAD			I	HUGH GREEN DRIVE	201449112	26/11/2014	Wed	550	KA	GSL	CW1M	302B 375B		R		F		X	G	C	60	0	0	0		
4124	GREAT SOUTH RD / PORTAGE RD	GREAT SOUTH ROAD			I	PORTAGE ROAD	201449448	19/12/2014	Fri	1640	KA	GSL	BN1C	302B		R	D	B	F	X	G	C	50	0	0	0		
2228	ST LUKES RD / CORNWALLIS RD / KINGSWAY AVE	ST LUKES ROAD			I	KINGSWAY AVENUE	201440458	25/07/2014	Fri	1405	KA	SCL	CW1V	322B		R	W	O	F	T	T	C	50	0	0	0		
3910	SH16 - LINCOLN RD WESTBOUND RAMPS	LINCOLN ROAD			I	LINCOLN OFF WBD	201431997	20/03/2014	Thu	957	KA	SSL	TS14	322A		R	D	BF	F	T	T	R	50	0	0	0		
2193	MT WELLINGTON HWAY / SYLVIA PARK RD	MOUNT WELLINGTON HIGHWAY			I	SYLVIA PARK ROAD	201442713	5/09/2014	Fri	1756	KA	SSL	CN1C	322A		R	W	DO	L	T	T	C	50	0	0	0		
4929	SH20 WESTBOUND RAMPS - LAMBIE DR	LAMBIE DRIVE			I	LAMBIE OFF WBD	201444729	29/08/2014	Fri	1140	KA	SSL	VS1C	322A		R	D	OF	F	T	T	N	50	0	0	0		
2251	MT WELLINGTON HWAY / ARANUI RD / SYLVIA PARK STORES	MOUNT WELLINGTON HIGHWAY			I	SILVIA PARK ACCESSWAY	201437183	26/04/2014	Sat	920	KA	ZSL	CS1C	302B 375B		R	D	OF	F	X	G	C	50	0	0	0		
1811	SH1 (HIBISCUS COAST HWAY) / EAST COAST ROAD	HIBISCUS COAST HIGHWAY			I	EAST COAST ROAD	201439757	13/06/2014	Fri	1746	KA	ZSL	CN2C	302B 375B		R	D	OO	F	X	G	R	50	0	0	0		
2222	GREAT SOUTH RD / CENTRAL PARK BUSINESS CENTRE	GREAT SOUTH ROAD			I	ACCESS ROAD	201440671	3/07/2014	Thu	1040	KA	ZSL	TS1C	322A 334A		R	W	O	F	T	T	C	50	0	0	0		
2027	COLLEGE HILL / JERVOIS RD / PONSONBY RD / ST MARYS RD / REDMOND ST	PONSONBY ROAD			I	COLLEGE HILL	201442700	30/07/2014	Wed	1717	KA	ZSL	CS1C	302B 375B		R	D	DO	F	X	G	C	50	0	0	0		
1208	NORTHCOTE RD - SUNNYBRAE RD - AKORANGA DR	NORTHCOTE ROAD			I	SUNNYBRAE ROAD	201448822	21/11/2014	Fri	810	KA	ZSL	CN1C	302B 375B		R	D	B	F	X	G	R	50	0	0	0		
4509	GT STH RD-CAVENDISH DR-TE IRIRANGI DR	GREAT SOUTH ROAD			I	TE IRIRANGI DRIVE	201434389	11/04/2014	Fri	147	KC	GSL	CS1C	103B 315B	P	R	W	DO	L	X	T	R	60	0	0	0		
1811	SH1 (HIBISCUS COAST HWAY) / EAST COAST ROAD	HIBISCUS COAST HIGHWAY			I	EAST COAST ROAD	201439758	13/06/2014	Fri	713	KC	GSL	TN2C	302A 375A		R	D	BF	F	X	G	C	80	0	0	0		
1645	SH17 - COLISEUM DR	COLISEUM DRIVE			I	ALBANY EXPRESSWAY	201443468	19/09/2014	Fri	958	KC	GSL	CE1B	302A 375A		E	W	B	F	M	G	C	50	0	0	0		
1413	GLENFIELD RD - CHIVALRY RD - MAYFIELD RD	CHIVALRY ROAD			I	GLENFIELD ROAD	201436341	28/03/2014	Fri	728	KC	ZSL	CS2C	302A 375A		E	D	OF	F	T	G	C	50	0	0	0		
2200	BALMORAL RD / MT EDEN RD	BALMORAL ROAD			I	MOUNT EDEN ROAD	201440675	22/07/2014	Tue	1700	KC	ZSL	CN2C	322B		E	D	O	F	X	T	R	50	0	0	0		

2008	QUEEN ST / UPPER QUEEN ST / KARANGAHAPE RD	KARANGAHAPE ROAD			I	UPPER QUEEN ST	201445845	14/10/2014	Tue	2140	KC	ZSL	VN2C	129A 386A 129B 386B		E	D	DO	F	X	G	L	50	0	0	0		
1811	SH1 (HIBISCUS COAST HWAY) / EAST COAST ROAD	HIBISCUS COAST HIGHWAY			I	BRIAN SMITH DRIVE	201447550	26/07/2014	Sat	1224	KC	ZSL	VE2V	302A 375A		R	D	B	F	X	G	R	80	0	0	0		
2008	QUEEN ST / UPPER QUEEN ST / KARANGAHAPE RD	KARANGAHAPE ROAD			I	UPPER QUEEN ST	201448954	1/12/2014	Mon	1800	KC	ZSL	BN2C	315B		E	D	B	F	X	G	C	50	0	0	0		
1407	GLENFIELD RD - KAIPATIKI RD	GLENFIELD ROAD			I	KAIPATIKI ROAD	201450577	9/02/2014	Sun	2212	KC	ZSL	CE2C	302A		R	D	DO	F	T	G	C	50	0	0	0		
4302	EAST TAMAKI RD - BAIRDS RD	EAST TAMAKI ROAD			I	BAIRDS ROAD	201450590	7/10/2014	Tue	2100	KC	ZSL	CS2C	375A		M	D	TO	F	X	T	C	60	0	0	0		
2378	GREAT NORTH RD / MCCRAE WAY	GREAT NORTH ROAD			I	MCCRAE WAY	201414936	22/06/2014	Sun	1906	NC	SCL	CN2E	341A 352A		R	D	DO	F	X	T	R	50	0	0	1	31	
2094	PITT ST / VINCENT ST	PITT ST			I	VINCENT ST	201438433	10/06/2014	Tue	803	NC	SCL	BS2E	323A 334A 901		M	W	O	H	X	T	C	50	0	0	0		
4137	CHURCH ST / PRINCES ST	CHURCH ST			I	PRINCES ST	201440960	15/08/2014	Fri	757	NC	SCL	CS1E	713B		R	D	OF	F	T	T	C	50	0	0	0		
1506	ONEWA RD - BIRKENHEAD AVE - HIGHBURY BYPASS	BIRKENHEAD AVENUE			I	ONEWA ROAD	201415995	10/09/2014	Wed	1545	NC	ZSL	CW2E	306A		M	D	B	F	X	T	X	50	0	0	1	13	
4102	MCKENZIE RD - MILLER RD - CORONATION RD - WALMSLEY RD	MCKENZIE ROAD			I	WALMSLEY ROAD	201412986	4/06/2014	Wed	620	NE	SCL	CE1E	718B 724B		R	D	DO	F	X	T	C	50	0	0	1	22	
4515	PUHINUI RD - CARRUTH RD - LAMBIE DR	PUHINUI ROAD			I	CARRUTH ROAD	201415628	5/08/2014	Tue	915	NE	SCL	CE1E	306A 363A 902		R	W	B	F	X	T	C	50	0	0	1		
2131	GILLIES AVE / EPSOM AVE	GILLIES AVENUE			I	EPSOM AVENUE	201416764	24/10/2014	Fri	1700	NE	SCL	CW2E	307A 330A		R	D	B	FS	X	T	C	50	0	0	1	28	

Appendix D List of the selected intersections

AREA	Intersection Number	Intersection Name	North Approach	Exclusive	Shared	Giveway	Signalised	Free Flow	Zebra	Raised Zebra	South Approach	Exclusive	Shared	Giveway	Signalised	Free Flow	Zebra	Raised Zebra	East Approach	Exclusive	Shared	Giveway	Signalised	Free Flow	Zebra	Raised Zebra	West Approach	Exclusive	Shared	Giveway	Signalised	Free Flow	Zebra	Raised Zebra	Intersection Type
North	1201	ANZAC ST - AUBURN ST	Auburn St	ECL							Auburn St				SSL				Anzac St	ECL							Anzac St		SCL						X
North	1212	TAHAROTA RD - SMALES FARM ENTRANCE									TAHAROTO RD						ZSL										SMALES FARM						ZSL		T
North	1216	NORTHCOTE RD - SMALES FARM - TAKAPUNA INT. SCHOOL	The Avenue (Smales)						ZSL																		Northcote rd						ZSL		T
North	1304	LAKE RD - JUTLAND RD - HAURAKI RD	Lake Rd						ZSL		Lake Rd		SCL						Hauraki Rd		SCL						Jutland Rd	ECL							X
North	1305	LAKE RD - ESMONDE RD									Lake Rd				SSL				Esmonde Rd	ECL															T
North	1402	GLENFIELD RD - MANUKU RD - HOGANS RD	GLENFIELD RD	ECL							GLENFIELD RD						ZSL		Hogans Rd	ECL							Manuku Rd	ECL							X
North	1409	GLENFIELD RD - ALBANY HWY - SUNSET RD - GLENDHU RD	Albany Hwy	ECL							Albany Hwy		SCL						Sunset Rd						ZSL		Glendhu Rd						ZSL		X
North	1413	GLENFIELD RD - CHIVALRY RD - MAYFIELD RD	GLENFIELD RD						ZSL		GLENFIELD RD		SCL						Chivalry Rd						ZSL		Mayfield Rd		SCL						X
North	1607	EAST COAST RD - BROWNS BAY RD	EAST COAST RD						ZSL										Browns Bay Rd				SSL												T
North	1617	ALBANY HWAY - OAKWAY DRIVE									Albany Hwy	ECL															Oakway Dr			GSL					T
North	1807	SH1 (HIBISCUS COAST HWAY) / WEST HOE RD									HIBISCUS COAST HWAY	ECL															WEST HOE RD			GSL					T
North	1810	SH1 (HIBISCUS COAST HWAY) / WHANGAPARAOA RD	HIBISCUS COAST HWAY			GSL					HIBISCUS COAST HWAY			GSL					WHANGAPARAOA RD					FSL			Millwater Parkway			GSL					X
North	1942	SH1 - ONEWA RD - SYLVAN AVE									Sylvan Ave					FSL											Onewa Rd			GSL					X
Central	2002	SYMONDS ST / KHYBER PASS RD / NEWTON RD	SYMONDS ST			GSL					SYMONDS ST	ECL							KHYBER PASS RD			GSL					NEWTON RD		SCL						X
Central	2008	QUEEN ST / UPPER QUEEN ST / KARANGAHAPE RD	QUEEN ST						ZSL		UPPER QUEEN ST						ZS		KARANGAHAPE RD		SC						KARANGAHAPE RD	EC							X

																	L				L							L							
Central	2013	GREEN LANE WEST / MANUKAU RD	MANUKAU RD		SCL						MANUKAU RD		SCL						GREEN LANE WEST						ZSL		GREEN LANE WEST	ECL							X
Central	2051	PONSONBY RD / RICHMOND RD / PICTON ST	PONSONBY RD		SCL						PONSONBY RD		SCL						PICTON ST		SCL						RICHMOND RD	ECL							X
Central	2086	GREEN LANE WEST / HOSPITAL GATE 3 / RACECOURSE CAR PARK	HOSPITAL GATE 3	ECL							RACECOURSE CAR PARK	ECL							GREEN LANE WEST		SCL						GREEN LANE WEST		SCL						X
Central	2094	PITT ST / VINCENT ST	PITT ST						ZSL		PITT ST						ZSL		VINCENT ST		SCL						Hopetoun St		SCL						X
Central	2109	ROSEBANK RD / ASH ST	ASH ST						ZSL		ASH ST			GSL					ROSEBANK RD		SCL						ROSEBANK RD		SCL						X
Central	2128	KOHIMARAMA RD / KEPA RD									KOHIMARAMA RD				SSL												KEPA RD			GSL					T
Central	2131	GILLIES AVE / EPSOM AVE	GILLIES AVE		SCL						GILLIES AVE		SCL						EPSOM AVE		SCL						EPSOM AVE		SCL						X
Central	2145	MT ALBERT RD / MT EDEN RD / WARREN AVE	MT EDEN RD		SCL						WARREN AVE		SCL						MT ALBERT RD		SCL						MT ALBERT RD		SCL						X
Central	2146	DOMINION RD / MT ALBERT RD	DOMINION RD		SCL						DOMINION RD		SCL						MT ALBERT RD		SCL						MT ALBERT RD		SCL						X
Central	2147	HILLSBOROUGH RD / HERD RD / CARR RD	HILLSBOROUGH RD		SCL						HILLSBOROUGH RD		SCL						HERD RD		SCL						CARR RD		SCL						X
Central	2153	NEW NORTH RD / MT ALBERT RD / CARRINGTON RD	CARRINGTON RD		SCL						MT ALBERT RD		SCL						NEW NORTH RD			GSL					NEW NORTH RD		SCL						X
Central	2155	MT ALBERT RD / SANDRINGHAM RD	SANDRINGHAM RD		SCL						SANDRINGHAM RD		SCL						MT ALBERT RD			GSL					MT ALBERT RD			GSL					X
Central	2159	MT SMART RD / ONEHUNGA MALL	ONEHUNGA MALL		SCL						ONEHUNGA MALL		SCL						MT SMART RD		SCL						MT SMART RD		SCL						X
Central	2166	BROADWAY / REMUERA RD	BROADWAY	ECL															REMUERA RD						ZSL										T
Central	2186	MT WELLINGTON HWAY / WAIPUNA RD / PENROSE RD	MT WELLINGTON HWAY		SCL						MT WELLINGTON HWAY						ZSL		WAIPUNA RD				SSL				PENROSE RD		SCL						X
Central	2190	GREAT SOUTH RD / CHURCH ST EAST									GREAT SOUTH RD						ZSL										CHURCH ST EAST						ZSL		T
Central	2193	MT WELLINGTON HWAY / SYLVIA PARK RD									MT WELLINGTON HWAY			GSL													SYLVIA PARK RD				SSL				T
Central	2196	PENROSE RD / BARRACK	BARRACK RD		SCL																						PENROSE RD		S						T

		RD																											CL						
Central	2197	CARBINE RD / PANAMA RD	CARBINE RD	ECL																							PANAMA RD		SCL						T
Central	2201	BALMORAL RD / DOMINION RD	DOMINION RD						ZSL		DOMINION RD						ZSL		BALMORAL RD						ZSL		BALMORAL RD						ZSL		X
Central	2204	NEW NORTH RD / ST LUKES RD	ST LUKES RD		SCL						ST LUKES RD						ZSL		NEW NORTH RD			GSL					NEW NORTH RD						ZSL		X
Central	2224	GREAT SOUTH RD / SYLVIA PARK RD	GREAT SOUTH RD					FSL											SYLVIA PARK RD			GSL													T
Central	2228	ST LUKES RD / CORNWALLIS RD / KINGSWAY AVE	CORNWALLIS RD	ECL							KINGSWAY AVE		SCL						ST LUKES RD		SCL						ST LUKES RD						ZSL		X
Central	2248	REMUERA RD / MIDDLETON RD	REMUERA RD		SCL														MIDDLETON RD	ECL															T
Central	2259	SEART / CARBINE RD	CARBINE RD		SCL						CARBINE RD					FSL			South Eastern Hwy	ECL							South Eastern Hwy			GSL					X
Central	2265	GREEN LANE EAST / LINK RD									Ellerslie Racecourse Dr				SSL				GREEN LANE EAST		SCL														T
Central	2287	ST HELIERS BAY RD / APIRANA AVE									APIRANA AVE					FSL			ST HELIERS BAY RD			GSL													T
Central	2307	MT WELLINGTON HWAY / SYLVIA PARK SHOPPING CENTRE	MT WELLINGTON HWAY						ZSL		MT WELLINGTON HWAY						ZSL		SYLVIA PARK SHOPPING CENTRE						ZSL		SYLVIA PARK SHOPPING CENTRE				SSL				X
Central	2308	CARBINE RD / SYLVIA PARK BUSINESS CENTRE									CARBINE RD		SCL														SYLVIA PARK BUSINESS CENTRE						ZSL		X
Central	2335	MAY RD / STODDARD RD / DENBIGH AVE	MAY RD		SCL						MAY RD	ECL							DENBIGH AVE		SCL						STODDARD RD		SCL						X
Central	2342	ABBOTTS WAY / LUNN AVE / NGAHUE DR	NGAHUE DR	ECL							NGAHUE DR		SCL						LUNN AVE		SCL						ABBOTTS WAY	ECL							X
Central	2354	CLARK ST / WARD ST	Ward St	ECL																							Clark St		SCL						T
Central	2378	GREAT NORTH RD / MCCRAE WAY	Driveway		SCL						MCCRAE WAY		SCL						GREAT NORTH RD		SCL						GREAT NORTH RD		SCL						X
Central	2905	PARNELL RISE / BEACH RD / STANLEY ST / THE STRAND	THE STRAND		SCL						STANLEY ST			GSL					PARNELL RISE	ECL							BEACH RD		SCL						X
West	3009	GREAT NORTH RD / WEST COAST RD									West Coast Rd	ECL							Great North Rd						ZSL										T

West	3010	GREAT NORTH RD / ARCHIBALD RD	Archibald Rd	ECL																							Great North Rd		SCL						T
West	3011	GREAT NORTH RD / GLENVIEW RD / SABULITE RD	Sabulite Rd		SCL						Glenview Rd	ECL							Great North Rd		SCL						Great North Rd		SCL						X
West	3017	GREAT NORTH RD / RAILSIDE AVE / RATANUI ST	Ratanui st	ECL							Railside Ave	ECL							Great North Rd		SCL						Great North Rd	ECL							X
West	3018	GREAT NORTH RD / HENDERSON VALLEY RD / ALDERMAN DRV	ALDERMAN DRV	ECL							Henderson Valley Rd	ECL							Great North Rd			GSL					Great North Rd			GSL					X
West	3020	LINCOLN RD - SEL PEACOCK DRV	Lincoln Rd		SCL														Sel Peacock Dr		SCL														T
West	3023	LINCOLN RD - UNIVERSAL DRV	Lincoln Rd			GSL					Lincoln Rd			GSL					Universal Dr		SCL						Universal Dr				SSL				X
West	3025	CENTRAL PARK DR - TE PAI PL										ECL															Te Pal Place		SCL						T
West	3074	FRED TAYLOR DR - WESTGATE SHOPPING									WESTGATE SHOPPING			GSL					FRED TAYLOR DR					FSL											T
South	4001	GEORGE BOLT MEM DR - TOM PEARCE DR	GEORGE BOLT MEM DR			GSL					GEORGE BOLT MEM DR			GSL					Tom Pearce Dr						ZSL		Tom Pearce Dr				SSL				X
South	4102	MCKENZIE RD - MILLER RD - CORONATION RD - WALMSLEY RD	Coronation Rd			GSL					Makenzie Rd			GSL					Walmsley Rd		SCL						Miller Rd		SCL						X
South	4109	MASSEY RD - ROBERTSON RD	Robertson Rd			GSL					Henwood Rd	ECL							MASSEY RD		SCL						MASSEY RD			GSL					X
South	4125	GREAT SOUTH RD / HUIA RD / ALBION ST	Great South Rd		SCL						Great South Rd		SCL						Albion St		SCL						Huia Rd	ECL							X
South	4126	ATKINSON AVE / PRINCES ST	Atkinson Ave						ZSL		Atkinson Ave						ZSL		Princes St		SCL						Princes St		SCL						X
South	4137	CHURCH ST / PRINCES ST	Church St		SCL						Church St		SCL						Princes St		SCL						Princes St		SCL						X
South	4205	PAKURANGA RD - CASCADES RD									Cascades						ZSL		Pakuranga Rd						ZSL										T
South	4219	TI RAKAU DR - BOTANY RD - TE IRIRANGI DR	BOTANY RD			GSL					BOTANY RD			GSL					Ti Rakau Dr			GSL					Ti Rakau Dr				SSL				X
South	4225	TI RAKAU DR - HARRIS RD									Harris Rd				SSL				Ti Rakau Dr			GSL													X
South	4233	CASCADES RD - BOTANY RD									Botany Dr					FSL			Cascades Dr			GSL													T

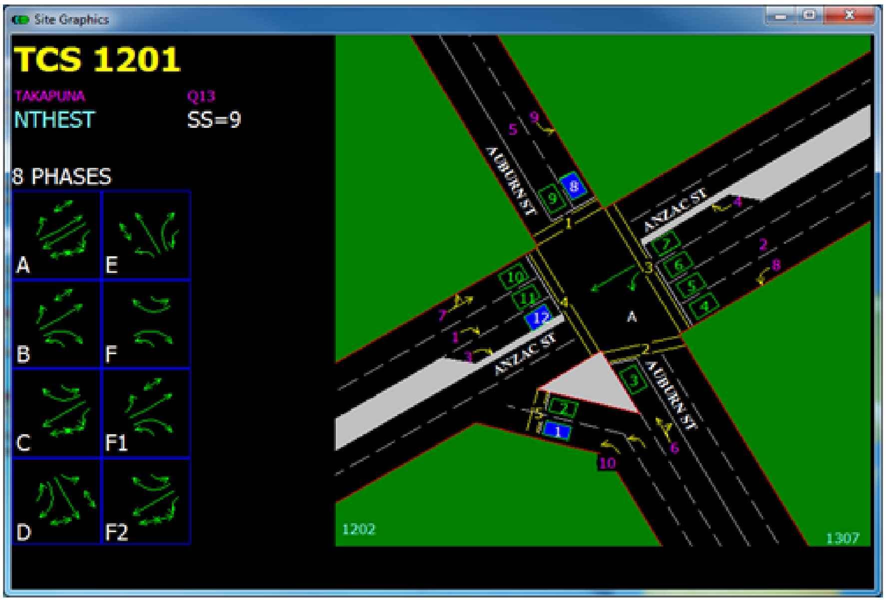
South	4300	EAST TAMAKI RD - FERGUSON RD									FERGUSON RD	ECL							East Tamaki Dr		SCL														T
South	4303	EAST TAMAKI RD - SPRINGS RD	SPRINGS RD			GSL					SPRINGS RD					FSL			East Tamaki Dr					FSL											T
South	4307	EAST TAMAKI RD - BIRMINGHAM RD	East Tamaki Dr		SCL														BIRMINGHAM RD	ECL															T
South	4323	CHAPEL RD - DAWSON RD	Chapel Rd		SCL						Matthews Rd		SCL						Thomas Rd			GSL					Dawson Rd		SCL						X
South	4330	HIGHBROOK DR - EL KOBAR DR									EL KOBAR DR				SSL				Highbrooke Dr			GSL													T
South	4331	BUSINESS PARADE STH - KERWYN DR	BUSINESS PARADE STH			GSL					BUSINESS PARADE STH				SSL				Highbrooke Dr			GSL					Highbrooke Dr			GSL					X
South	4334	BAIRDS RD - HELLABYS RD	HELLABYS RD				SSL																				Bairds Rd					FSL			T
South	4403	GT STH RD - BROWNS RD - ORAMS RD	Great South Rd			GSL					Great South Rd		SCL						Orams Rd			GSL					Browns Rd			GSL					X
South	4404	KERRS RD - DALGETY DR									Dalgey Dr	ECL							Kerrs RD	ECL															T
South	4406	GT STH RD - MAHIA RD									Great South Rd			GSL													Mahia Rd		SCL						T
South	4515	PUHINUI RD - CARRUTH RD - LAMBIE DR	CARRUTH RD		SCL						LAMBIE DR		SCL						PUHINUI RD			GSL					PUHINUI RD		SCL						X
South	4530	WIRI STATION RD - ROSCOMMON OIL TERMINAL	ROSCOMMON RD					FSL			ROSCOMMON RD		SCL						WIRI STATION RD			GSL					WIRI STATION RD			GSL					X
South	4608	TE IRIRANGI DR TOWN CENTRE DR - TE KOHA RD	TE IRIRANGI DR					FSL			TE IRIRANGI DR			GSL					TE KOHA RD			GSL					TOWN CENTRE DR			GSL					X
South	4612	ORMISTON RD - MURPHYS RD	MURPHYS RD			GSL					MURPHYS RD			GSL					ORMISTON RD			GSL					ORMISTON RD			GSL					X
South	4903	KIRKBRIDE RD - GEORGE BOLT MEM - SH 20A	GEORGE BOLT MEM - SH 20A				SSL				GEORGE BOLT MEM - SH 20A			GSL					KIRKBRIDE RD					FSL			KIRKBRIDE RD				SSL				X
South	4905	GEORGE BOLT MEM DR - MONTGOMERY RD									GEORGE BOLT MEM DR			GSL													MONTGOMERY RD					FSL			T

Appendix E A sample site of SCATS® detector loop counts

Volume Data

Site	120												
Date	**Date**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
01-03-2016		3640	1317	1296	1519	4428	2605	318	255	1323	5821	1901	1856
02-03-2016		3522	1277	1303	1491	4402	2629	325	224	1270	5791	1826	1812
03-03-2016		3618	1348	1323	1608	4566	2862	309	213	1345	6251	1985	1915
04-03-2016		3545	1185	1249	1378	4575	2891	295	254	1237	6399	1890	1854
05-03-2016		2127	565	883	1084	3968	2423	277	126	691	5768	1393	1254
06-03-2016		2069	558	837	1017	3703	2248	263	136	583	5607	1355	1178
07-03-2016		3275	1183	1109	1273	4080	2293	311	206	1173	5343	1677	1775
08-03-2016		3431	1284	1229	1399	4461	2634	326	259	1294	5972	1806	1812
09-03-2016		3507	1307	1224	1396	4459	2635	272	209	1264	5956	1792	1814
10-03-2016		3633	1303	1339	1524	4556	2745	305	197	1284	6106	1989	1894
11-03-2016		3517	1204	1230	1463	4495	2786	277	215	1289	6366	2012	1937
12-03-2016		2296	546	906	1098	4004	2496	296	189	793	5983	1424	1221
13-03-2016		2137	552	977	1071	3880	2495	285	288	936	6075	1370	1123
14-03-2016		3291	1180	1171	1345	4049	2495	268	220	1163	5324	1677	1687
15-03-2016		3399	1238	1264	1379	4380	2677	306	219	1257	5903	1746	1786
16-03-2016		3553	1303	1282	1440	4449	2654	349	259	1304	5896	1891	1841
17-03-2016		3664	1303	1384	1684	4702	2985	399	265	1402	6853	2055	1931
18-03-2016		3416	1167	1213	1446	4554	2793	298	237	1212	6131	1800	1747
19-03-2016		2220	556	870	1135	3775	2390	266	119	678	5491	1463	1230
20-03-2016		2049	495	812	1013	3587	2248	305	174	616	5154	1396	1071
21-03-2016		3414	1190	1177	1295	4066	2450	286	224	1196	5397	1699	1776
22-03-2016		3474	1293	1281	1409	4357	2599	301	248	1311	5793	1796	1770
23-03-2016		3815	1421	1192	1505	4451	2606	337	219	1308	5655	1948	1857
24-03-2016		3946	1298	1290	1598	4739	2892	338	175	1195	6444	2065	2001
25-03-2016		1032	241	419	409	2562	1687	143	89	314	3902	659	614
26-03-2016		1868	444	623	963	3431	2033	253	93	500	4912	1270	1080
27-03-2016		1246	260	519	610	2947	1880	157	105	366	4708	849	782
28-03-2016		1584	378	540	847	3029	1841	241	78	457	4348	1093	925
29-03-2016		3249	1170	1103	1424	4290	2608	321	177	1061	5973	1945	1811
		2851	952	1035	1228	3965	2420	281	189	994	5511	1593	1512
Average		**2946**	**984**	**1069**	**1268**	**4097**	**2500**	**290**	**195**	**1027**	**5694**	**1645**	**1562**

Approach Name	Direction	LT Type	Traffic Volume Vpd
Auburn St	N	ECL	1223
Auburn St	S	SSL	5000
Anzac St	E	ECL	8156
Anzac St	W	SCL	8902



Appendix F Results of Method A (MA): using the existing traffic volumes

Appendix F.1 Results of base model: give-way left turn slip lanes

INTERSECTION SUMMARY

Site: Site 1644: Albany Expressway - Massey University -Base

Base Model - Slip lane with give-way control
Signals - Fixed Time   Cycle Time = 120 seconds (User-Given Cycle Time)

Intersection Performance - Hourly Values						
Performance Measure	**Vehicles**		**Pedestrians**		**Persons**	
Travel Speed (Average)	45.6	km/h	2.0	km/h	44.3	km/h
Travel Distance (Total)	2492.6	veh-km/h	4.1	ped-km/h	2995.2	pers-km/h
Travel Time (Total)	54.7	veh-h/h	2.0	ped-h/h	67.6	pers-h/h
Demand Flows (Total)	2443	veh/h	105	ped/h	2932	pers/h
Percent Heavy Vehicles (Demand)	5.0	%				
Degree of Saturation	0.866		0.088			
Practical Spare Capacity	4.0	%				
Effective Intersection Capacity	2822	veh/h				
Control Delay (Total)	12.80	veh-h/h	1.06	ped-h/h	16.41	pers-h/h
Control Delay (Average)	18.9	sec	36.2	sec	20.2	sec
Control Delay (Worst Lane)	73.9	sec				
Control Delay (Worst Movement)	73.9	sec	54.3	sec	73.9	sec
Geometric Delay (Average)	2.2	sec				
Stop-Line Delay (Average)	16.6	sec				
Idling Time (Average)	14.5	sec				
Intersection Level of Service (LOS)	LOS B		LOS D			
95% Back of Queue - Vehicles (Worst Lane)	10.5	veh				
95% Back of Queue - Distance (Worst Lane)	76.9	m				
Queue Storage Ratio (Worst Lane)	0.09					
Total Effective Stops	1103	veh/h	79	ped/h	1402	pers/h
Effective Stop Rate	0.45	per veh	0.75	per ped	0.48	per pers
Proportion Queued	0.43		0.75		0.46	
Performance Index	106.3		2.4		108.7	
Cost (Total)	1430.36	$/h	45.74	$/h	1476.09	$/h
Fuel Consumption (Total)	230.6	L/h				
Carbon Dioxide (Total)	549.1	kg/h				
Hydrocarbons (Total)	0.174	kg/h				
Carbon Monoxide (Total)	2.414	kg/h				
NOx (Total)	1.224	kg/h				

Level of Service (LOS) Method: Delay (HCM 2000).
Intersection LOS value for Vehicles is based on average delay for all vehicle movements.
Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.
SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.

Intersection Performance - Annual Values						
Performance Measure	**Vehicles**		**Pedestrians**		**Persons**	
Demand Flows (Total)	1,172,716	veh/y	50,526	ped/y	1,407,259	pers/y
Delay	6,142	veh-h/y	508	ped-h/y	7,879	pers-h/y
Effective Stops	529,225	veh/y	37,978	ped/y	673,048	pers/y
Travel Distance	1,196,430	veh-km/y	1,963	ped-km/y	1,437,679	pers-km/y
Travel Time	26,257	veh-h/y	963	ped-h/y	32,472	pers-h/y

Cost	686,571	$/y	21,953	$/y	708,525	$/y
Fuel Consumption	110,705	L/y				
Carbon Dioxide	263,553	kg/y				
Hydrocarbons	84	kg/y				
Carbon Monoxide	1,159	kg/y				
NOx	588	kg/y				

MOVEMENT SUMMARY

Site: Site 1644: Albany Expressway - Massey University – base model

Base Model - Slip lane with give-way control
Signals - Fixed Time Cycle Time = 120 seconds (User-Given Cycle Time)

Movement Performance - Vehicles

Mov ID	ODMov	Demand Flows Total	Demand Flows HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
SouthEast: Albany Expressway S											
4	L2	151	5.0	0.139	7.2	LOS A	1.3	9.4	0.22	0.60	52.9
5	T1	837	5.0	0.469	11.9	LOS B	10.0	72.9	0.44	0.39	50.2
Approach		987	5.0	0.469	11.2	LOS B	10.0	72.9	0.40	0.42	50.6
NorthWest: Albany Expressway N											
11	T1	645	5.0	0.233	0.8	LOS A	0.8	5.9	0.05	0.04	59.2
12	R2	143	5.0	0.866	73.9	LOS E	9.5	69.4	1.00	0.97	26.8
Approach		788	5.0	0.866	14.1	LOS B	9.5	69.4	0.22	0.21	48.6
SouthWest: Massey University											
1	L2	317	5.0	0.405	9.2	LOS A	5.6	40.7	0.39	0.68	51.4
3	R2	351	5.0	0.728	60.0	LOS E	10.5	76.9	1.00	0.87	29.9
Approach		667	5.0	0.728	35.9	LOS D	10.5	76.9	0.71	0.78	37.3
All Vehicles		2443	5.0	0.866	18.9	LOS B	10.5	76.9	0.43	0.45	45.6

Level of Service (LOS) Method: Delay (HCM 2000).
Vehicle movement LOS values are based on average delay per movement
Intersection and Approach LOS values are based on average delay for all vehicle movements.
SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Movement Performance - Pedestrians

Mov ID	Description	Demand Flow	Average Delay	Level of Service	Average Back of Queue Pedestrian	Average Back of Queue Distance	Prop. Queued	Effective Stop Rate
		ped/h	sec		ped	m		per ped
P2	NorthWest Full Crossing	53	54.3	LOS E	0.2	0.2	0.95	0.95
P1	SouthWest Full Crossing	53	18.2	LOS B	0.1	0.1	0.55	0.55
All Pedestrians		105	36.2	LOS D			0.75	0.75

Level of Service (LOS) Method: SIDRA Pedestrian LOS Method (Based on Average Delay)
Pedestrian movement LOS values are based on average delay per pedestrian movement.
Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.

Appendix F.2 Results of Option 1: signalised left turn slip lanes

INTERSECTION SUMMARY

Site: Site 1644: Albany Expressway - Massey University - Option 1_SSL

Option 1 - Signalised left turn slip lane
Signals - Fixed Time   Cycle Time = 120 seconds (User-Given Cycle Time)

Intersection Performance - Hourly Values

Performance Measure	Vehicles		Pedestrians		Persons	
Travel Speed (Average)	37.9	km/h	1.5	km/h	36.0	km/h
Travel Distance (Total)	2492.6	veh-km/h	6.6	ped-km/h	2997.7	pers-km/h
Travel Time (Total)	65.8	veh-h/h	4.4	ped-h/h	83.3	pers-h/h
Demand Flows (Total)	2443	veh/h	211	ped/h	2932	pers/h
Percent Heavy Vehicles (Demand)	5.0	%				
Degree of Saturation	0.914		0.088			
Practical Spare Capacity	-1.5	%				
Effective Intersection Capacity	2674	veh/h				
Control Delay (Total)	23.86	veh-h/h	2.85	ped-h/h	31.49	pers-h/h
Control Delay (Average)	35.2	sec	48.8	sec	38.7	sec
Control Delay (Worst Lane)	70.8	sec				
Control Delay (Worst Movement)	70.8	sec	54.3	sec	70.8	sec
Geometric Delay (Average)	2.2	sec				
Stop-Line Delay (Average)	32.9	sec				
Idling Time (Average)	29.5	sec				
Intersection Level of Service (LOS)	LOS D		LOS E			
95% Back of Queue - Vehicles (Worst Lane)	22.1	veh				
95% Back of Queue - Distance (Worst Lane)	161.1	m				
Queue Storage Ratio (Worst Lane)	0.17					
Total Effective Stops	1514	veh/h	189	ped/h	2006	pers/h
Effective Stop Rate	0.62	per veh	0.90	per ped	0.68	per pers
Proportion Queued	0.68		0.90		0.74	
Performance Index	156.7		5.4		162.1	
Cost (Total)	1831.29	$/h	99.61	$/h	1930.90	$/h
Fuel Consumption (Total)	257.8	L/h				
Carbon Dioxide (Total)	613.3	kg/h				
Hydrocarbons (Total)	0.212	kg/h				
Carbon Monoxide (Total)	2.619	kg/h				
NOx (Total)	1.377	kg/h				

Level of Service (LOS) Method: Delay (HCM 2000).
Intersection LOS value for Vehicles is based on average delay for all vehicle movements.
Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.
SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.

Intersection Performance - Annual Values

Performance Measure	Vehicles		Pedestrians		Persons	
Demand Flows (Total)	1,172,716	veh/y	101,053	ped/y	1,407,259	pers/y
Delay	11,455	veh-h/y	1,369	ped-h/y	15,115	pers-h/y
Effective Stops	726,742	veh/y	90,725	ped/y	962,816	pers/y
Travel Distance	1,196,430	veh-km/y	3,191	ped-km/y	1,438,907	pers-km/y

Travel Time	31,566	veh-h/y	2,097	ped-h/y	39,976	pers-h/y
Cost	879,019	$/y	47,814	$/y	926,833	$/y
Fuel Consumption	123,751	L/y				
Carbon Dioxide	294,394	kg/y				
Hydrocarbons	102	kg/y				
Carbon Monoxide	1,257	kg/y				
NOx	661	kg/y				

SIDRA INTERSECTION 6

MOVEMENT SUMMARY

Site: Site 1644: Albany Expressway - Massey University - Option 1_SSL

Option 1 - Signalised left turn slip lane
Signals - Fixed Time Cycle Time = 120 seconds (User-Given Cycle Time)

Movement Performance - Vehicles

Mov ID	ODMo v	Demand Flows Total	Demand Flows HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
SouthEast: Albany Expressway S											
4	L2	151	5.0	0.668	58.3	LOS E	8.7	63.4	0.99	0.85	30.5
5	T1	837	5.0	0.703	32.5	LOS C	19.4	141.6	0.84	0.74	39.2
Approach		987	5.0	0.703	36.4	LOS D	19.4	141.6	0.86	0.75	37.6
NorthWest: Albany Expressway N											
11	T1	645	5.0	0.233	0.8	LOS A	0.8	5.9	0.05	0.04	59.2
12	R2	143	5.0	0.322	41.6	LOS D	6.6	48.1	0.83	0.78	35.2
Approach		788	5.0	0.322	8.2	LOS A	6.6	48.1	0.19	0.17	52.7
SouthWest: Massey University											
1	L2	317	5.0	0.914	70.8	LOS E	22.1	161.1	0.98	1.03	27.6
3	R2	351	5.0	0.728	60.0	LOS E	10.5	76.9	1.00	0.87	29.9
Approach		667	5.0	0.914	65.2	LOS E	22.1	161.1	0.99	0.95	28.8
All Vehicles		2443	5.0	0.914	35.2	LOS D	22.1	161.1	0.68	0.62	37.9

Level of Service (LOS) Method: Delay (HCM 2000).
Vehicle movement LOS values are based on average delay per movement
Intersection and Approach LOS values are based on average delay for all vehicle movements.
SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Movement Performance - Pedestrians

Mov ID	Description	Demand Flow	Average Delay	Level of Service	Average Back of Queue Pedestrian	Average Back of Queue Distance	Prop. Queued	Effective Stop Rate
		ped/h	sec		ped	m		per ped
P3	SouthEast Slip/Bypass Lane Crossing	53	54.3	LOS E	0.2	0.2	0.95	0.95
P2	NorthWest Full Crossing	53	54.3	LOS E	0.2	0.2	0.95	0.95
P1	SouthWest Full Crossing	53	32.3	LOS D	0.1	0.1	0.73	0.73
P4	SouthWest Slip/Bypass Lane Crossing	53	54.3	LOS E	0.2	0.2	0.95	0.95
All Pedestrians		211	48.8	LOS E			0.90	0.90

Level of Service (LOS) Method: SIDRA Pedestrian LOS Method (Based on Average Delay)
Pedestrian movement LOS values are based on average delay per pedestrian movement.
Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.

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Appendix F.3 Results of Option 2A: exclusive left turn lanes with pedestrian protection for the whole walk time

INTERSECTION SUMMARY

Site: Site 1644: Albany Expressway - Massey University - Option 2A_ECL

Option 2A: exclusive left turn lane with protection for the whole walk time
Signals - Fixed Time Cycle Time = 120 seconds (User-Given Cycle Time)

Intersection Performance - Hourly Values			
Performance Measure	**Vehicles**	**Pedestrians**	**Persons**
Travel Speed (Average)	42.0 km/h	1.7 km/h	40.6 km/h
Travel Distance (Total)	2488.9 veh-km/h	4.3 ped-km/h	2990.9 pers-km/h
Travel Time (Total)	59.2 veh-h/h	2.6 ped-h/h	73.7 pers-h/h
Demand Flows (Total)	2443 veh/h	105 ped/h	2932 pers/h
Percent Heavy Vehicles (Demand)	5.0 %		
Degree of Saturation	0.866	0.088	
Practical Spare Capacity	4.0 %		
Effective Intersection Capacity	2822 veh/h		
Control Delay (Total)	17.30 veh-h/h	1.59 ped-h/h	22.34 pers-h/h
Control Delay (Average)	25.5 sec	54.3 sec	27.4 sec
Control Delay (Worst Lane)	73.9 sec		
Control Delay (Worst Movement)	73.9 sec	54.3 sec	73.9 sec
Geometric Delay (Average)	2.2 sec		
Stop-Line Delay (Average)	23.3 sec		
Idling Time (Average)	20.7 sec		
Intersection Level of Service (LOS)	LOS C	LOS E	
95% Back of Queue - Vehicles (Worst Lane)	19.1 veh		
95% Back of Queue - Distance (Worst Lane)	139.7 m		
Queue Storage Ratio (Worst Lane)	0.10		
Total Effective Stops	1261 veh/h	100 ped/h	1613 pers/h
Effective Stop Rate	0.52 per veh	0.95 per ped	0.55 per pers
Proportion Queued	0.54	0.95	0.58
Performance Index	132.2	3.1	135.3
Cost (Total)	1588.65 $/h	58.68 $/h	1647.33 $/h
Fuel Consumption (Total)	240.6 L/h		
Carbon Dioxide (Total)	572.7 kg/h		
Hydrocarbons (Total)	0.189 kg/h		
Carbon Monoxide (Total)	2.490 kg/h		
NOx (Total)	1.278 kg/h		

Level of Service (LOS) Method: Delay (HCM 2000).
Intersection LOS value for Vehicles is based on average delay for all vehicle movements.
Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.
SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.

Intersection Performance - Annual Values						
Performance Measure	**Vehicles**		**Pedestrians**		**Persons**	
Demand Flows (Total)	1,172,716	veh/y	50,526	ped/y	1,407,259	pers/y
Delay	8,302	veh-h/y	762	ped-h/y	10,725	pers-h/y
Effective Stops	605,045	veh/y	48,105	ped/y	774,159	pers/y
Travel Distance	1,194,668	veh-km/y	2,046	ped-km/y	1,435,648	pers-km/y
Travel Time	28,439	veh-h/y	1,235	ped-h/y	35,362	pers-h/y
Cost	762,552	$/y	28,166	$/y	790,718	$/y
Fuel Consumption	115,497	L/y				
Carbon Dioxide	274,881	kg/y				
Hydrocarbons	91	kg/y				
Carbon Monoxide	1,195	kg/y				
NOx	613	kg/y				

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MOVEMENT SUMMARY

Site: Site 1644: Albany Expressway - Massey University - Option 2A_ECL

Option 2A: exclusive left turn lane with protection for the whole walk time

Signals - Fixed Time Cycle Time = 120 seconds (User-Given Cycle Time)

Movement Performance - Vehicles											
Mov ID	ODMov	Demand Flows		Deg. Satn	Average Delay	Level of Service	95% Back of Queue		Prop. Queued	Effective Stop Rate	Average Speed
		Total	HV				Vehicles	Distance			
		veh/h	%	v/c	sec		veh	m		per veh	km/h
SouthEast: Albany Expressway S											
4	L2	151	5.0	0.159	10.7	LOS B	2.6	19.1	0.32	0.66	49.7
5	T1	837	5.0	0.499	14.9	LOS B	11.7	85.2	0.51	0.45	48.2
Approach		987	5.0	0.499	14.3	LOS B	11.7	85.2	0.48	0.48	48.5
NorthWest: Albany Expressway N											
11	T1	645	5.0	0.244	2.1	LOS A	1.9	13.7	0.11	0.09	58.0
12	R2	143	5.0	0.866	73.9	LOS E	9.5	69.4	1.00	0.97	26.8
Approach		788	5.0	0.866	15.1	LOS B	9.5	69.4	0.27	0.25	47.9
SouthWest: Massey University											
1	L2	317	5.0	0.838	55.5	LOS E	19.1	139.7	0.96	0.93	30.9
3	R2	351	5.0	0.612	53.3	LOS D	9.7	70.9	0.96	0.82	31.6
Approach		667	5.0	0.838	54.3	LOS D	19.1	139.7	0.96	0.87	31.3
All Vehicles		2443	5.0	0.866	25.5	LOS C	19.1	139.7	0.54	0.52	42.0

Level of Service (LOS) Method: Delay (HCM 2000).
Vehicle movement LOS values are based on average delay per movement
Intersection and Approach LOS values are based on average delay for all vehicle movements.
SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Movement Performance - Pedestrians								
Mov ID	Description	Demand Flow	Average Delay	Level of Service	Average Back of Queue		Prop. Queued	Effective Stop Rate
					Pedestrian	Distance		
		ped/h	sec		ped	m		per ped
P2	NorthWest Full Crossing	53	54.3	LOS E	0.2	0.2	0.95	0.95
P1	SouthWest Full Crossing	53	54.3	LOS E	0.2	0.2	0.95	0.95
All Pedestrians		105	54.3	LOS E			0.95	0.95

Level of Service (LOS) Method: SIDRA Pedestrian LOS Method (Based on Average Delay)

Pedestrian movement LOS values are based on average delay per pedestrian movement.
Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.

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Appendix F.4 Results of Option 2B: exclusive left turn lanes with pedestrian protection for the whole walk time and the half clearance time

INTERSECTION SUMMARY

Site: Site 1644: Albany Expressway - Massey University - Option 2B_ECL

Option 2B: exclusive left turn lane with protection for walk and half clearance intervals
Signals - Fixed Time Cycle Time = 120 seconds (User-Given Cycle Time)

Intersection Performance - Hourly Values						
Performance Measure	Vehicles		Pedestrians		Persons	
Travel Speed (Average)	40.8	km/h	1.7	km/h	39.5	km/h
Travel Distance (Total)	2488.9	veh-km/h	4.3	ped-km/h	2990.9	pers-km/h
Travel Time (Total)	61.0	veh-h/h	2.6	ped-h/h	75.7	pers-h/h
Demand Flows (Total)	2443	veh/h	105	ped/h	2932	pers/h
Percent Heavy Vehicles (Demand)	5.0	%				
Degree of Saturation	0.893		0.088			
Practical Spare Capacity	0.7	%				
Effective Intersection Capacity	2734	veh/h				
Control Delay (Total)	19.01	veh-h/h	1.59	ped-h/h	24.40	pers-h/h
Control Delay (Average)	28.0	sec	54.3	sec	30.0	sec
Control Delay (Worst Lane)	65.9	sec				
Control Delay (Worst Movement)	65.9	sec	54.3	sec	65.9	sec
Geometric Delay (Average)	2.2	sec				
Stop-Line Delay (Average)	25.8	sec				
Idling Time (Average)	23.0	sec				
Intersection Level of Service (LOS)	LOS C		LOS E			
95% Back of Queue - Vehicles (Worst Lane)	21.2	veh				
95% Back of Queue - Distance (Worst Lane)	154.8	m				
Queue Storage Ratio (Worst Lane)	0.13					
Total Effective Stops	1342	veh/h	100	ped/h	1710	pers/h
Effective Stop Rate	0.55	per veh	0.95	per ped	0.58	per pers
Proportion Queued	0.59		0.95		0.63	
Performance Index	139.7		3.1		142.8	
Cost (Total)	1657.08	$/h	58.68	$/h	1715.76	$/h
Fuel Consumption (Total)	245.9	L/h				
Carbon Dioxide (Total)	585.3	kg/h				
Hydrocarbons (Total)	0.195	kg/h				
Carbon Monoxide (Total)	2.529	kg/h				
NOx (Total)	1.312	kg/h				

Level of Service (LOS) Method: Delay (HCM 2000).
Intersection LOS value for Vehicles is based on average delay for all vehicle movements.
Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.
SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.

Intersection Performance - Annual Values						
Performance Measure	Vehicles		Pedestrians		Persons	
Demand Flows (Total)	1,172,716	veh/y	50,526	ped/y	1,407,259	pers/y
Delay	9,127	veh-h/y	762	ped-h/y	11,714	pers-h/y
Effective Stops	643,936	veh/y	48,105	ped/y	820,829	pers/y
Travel Distance	1,194,668	veh-km/y	2,046	ped-km/y	1,435,648	pers-km/y
Travel Time	29,264	veh-h/y	1,235	ped-h/y	36,352	pers-h/y
Cost	795,399	$/y	28,166	$/y	823,565	$/y
Fuel Consumption	118,054	L/y				
Carbon Dioxide	280,926	kg/y				
Hydrocarbons	94	kg/y				
Carbon Monoxide	1,214	kg/y				
NOx	630	kg/y				

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MOVEMENT SUMMARY

Site: Site 1644: Albany Expressway - Massey University - Option 2B_ECL

Option 2B: exclusive left turn lane with protection for walk and half clearance intervals

Signals - Fixed Time Cycle Time = 120 seconds (User-Given Cycle Time)

Movement Performance - Vehicles											
Mov ID	ODMo v	Demand Flows Total	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
SouthEast: Albany Expressway S											
4	L2	151	5.0	0.192	16.7	LOS B	3.9	28.2	0.47	0.70	46.0
5	T1	837	5.0	0.553	20.0	LOS C	14.2	103.8	0.62	0.55	45.2
Approach		987	5.0	0.553	19.5	LOS B	14.2	103.8	0.60	0.57	45.3
NorthWest: Albany Expressway N											
11	T1	645	5.0	0.244	2.1	LOS A	1.9	13.7	0.11	0.09	58.0
12	R2	143	5.0	0.592	57.7	LOS E	8.0	58.7	0.98	0.81	30.4
Approach		788	5.0	0.592	12.2	LOS B	8.0	58.7	0.27	0.22	49.8
SouthWest: Massey University											
1	L2	317	5.0	0.893	65.9	LOS E	21.2	154.8	0.98	0.99	28.4
3	R2	351	5.0	0.612	53.3	LOS D	9.7	70.9	0.96	0.82	31.6
Approach		667	5.0	0.893	59.3	LOS E	21.2	154.8	0.97	0.90	30.0
All Vehicles		2443	5.0	0.893	28.0	LOS C	21.2	154.8	0.59	0.55	40.8

Level of Service (LOS) Method: Delay (HCM 2000).
Vehicle movement LOS values are based on average delay per movement
Intersection and Approach LOS values are based on average delay for all vehicle movements.
SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Movement Performance - Pedestrians								
Mov ID	Description	Demand Flow	Average Delay	Level of Service	Average Back of Queue Pedestrian	Average Back of Queue Distance	Prop. Queued	Effective Stop Rate
		ped/h	sec		ped	m		per ped
P2	NorthWest Full Crossing	53	54.3	LOS E	0.2	0.2	0.95	0.95
P1	SouthWest Full Crossing	53	54.3	LOS E	0.2	0.2	0.95	0.95
All Pedestrians		105	54.3	LOS E			0.95	0.95

Level of Service (LOS) Method: SIDRA Pedestrian LOS Method (Based on Average Delay)
Pedestrian movement LOS values are based on average delay per pedestrian movement.
Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.

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Appendix F.5 Results of Option 2C: exclusive left turn lanes with pedestrian protection for the whole walk time and the full clearance time

INTERSECTION SUMMARY

Site: Site 1644: Albany Expressway - Massey University -Option 2C_ECL

Option 2C: Exclusive left turn lane with protection for walk and full clearance intervals
Signals - Fixed Time Cycle Time = 120 seconds (User-Given Cycle Time)

Intersection Performance - Hourly Values						
Performance Measure	**Vehicles**		**Pedestrians**		**Persons**	
Travel Speed (Average)	39.1	km/h	1.7	km/h	37.9	km/h
Travel Distance (Total)	2488.9	veh-km/h	4.3	ped-km/h	2990.9	pers-km/h
Travel Time (Total)	63.6	veh-h/h	2.6	ped-h/h	78.9	pers-h/h
Demand Flows (Total)	2443	veh/h	105	ped/h	2932	pers/h
Percent Heavy Vehicles (Demand)	5.0	%				
Degree of Saturation	0.903		0.088			
Practical Spare Capacity	-0.3	%				
Effective Intersection Capacity	2706	veh/h				
Control Delay (Total)	21.65	veh-h/h	1.59	ped-h/h	27.56	pers-h/h
Control Delay (Average)	31.9	sec	54.3	sec	33.8	sec
Control Delay (Worst Lane)	68.6	sec				
Control Delay (Worst Movement)	68.6	sec	54.3	sec	68.6	sec
Geometric Delay (Average)	2.2	sec				
Stop-Line Delay (Average)	29.7	sec				
Idling Time (Average)	26.5	sec				
Intersection Level of Service (LOS)	LOS C		LOS E			
95% Back of Queue - Vehicles (Worst Lane)	21.7	veh				
95% Back of Queue - Distance (Worst Lane)	158.5	m				
Queue Storage Ratio (Worst Lane)	0.18					
Total Effective Stops	1530	veh/h	100	ped/h	1936	pers/h
Effective Stop Rate	0.63	per veh	0.95	per ped	0.66	per pers
Proportion Queued	0.68		0.95		0.71	
Performance Index	151.6		3.1		154.7	
Cost (Total)	1769.79	$/h	58.68	$/h	1828.47	$/h
Fuel Consumption (Total)	256.3	L/h				
Carbon Dioxide (Total)	609.8	kg/h				
Hydrocarbons (Total)	0.207	kg/h				
Carbon Monoxide (Total)	2.603	kg/h				
NOx (Total)	1.387	kg/h				

Level of Service (LOS) Method: Delay (HCM 2000).
Intersection LOS value for Vehicles is based on average delay for all vehicle movements.
Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.
SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.

Intersection Performance - Annual Values						
Performance Measure	Vehicles		Pedestrians		Persons	
Demand Flows (Total)	1,172,716	veh/y	50,526	ped/y	1,407,259	pers/y
Delay	10,390	veh-h/y	762	ped-h/y	13,229	pers-h/y
Effective Stops	734,423	veh/y	48,105	ped/y	929,413	pers/y
Travel Distance	1,194,668	veh-km/y	2,046	ped-km/y	1,435,648	pers-km/y
Travel Time	30,526	veh-h/y	1,235	ped-h/y	37,867	pers-h/y
Cost	849,498	$/y	28,166	$/y	877,664	$/y
Fuel Consumption	123,036	L/y				
Carbon Dioxide	292,705	kg/y				
Hydrocarbons	100	kg/y				
Carbon Monoxide	1,249	kg/y				
NOx	666	kg/y				

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MOVEMENT SUMMARY

Site: Site 1644: Albany Expressway - Massey University - Option 2C_ECL

Option 2C: Exclusive left turn lane with protection for walk and full clearance intervals
Signals - Fixed Time Cycle Time = 120 seconds (User-Given Cycle Time)

Movement Performance - Vehicles											
Mov ID	ODMov	Demand Flows Total	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
SouthEast: Albany Expressway S											
4	L2	151	5.0	0.246	25.7	LOS C	5.2	38.1	0.63	0.74	41.3
5	T1	837	5.0	0.699	30.2	LOS C	19.6	143.4	0.80	0.70	40.2
Approach		987	5.0	0.699	29.5	LOS C	19.6	143.4	0.77	0.71	40.3
NorthWest: Albany Expressway N											
11	T1	645	5.0	0.263	4.5	LOS A	3.5	25.5	0.20	0.17	55.9
12	R2	143	5.0	0.489	53.3	LOS D	7.6	55.8	0.94	0.80	31.6
Approach		788	5.0	0.489	13.4	LOS B	7.6	55.8	0.33	0.29	49.0
SouthWest: Massey University											
1	L2	317	5.0	0.903	68.6	LOS E	21.7	158.5	0.99	1.00	27.8
3	R2	351	5.0	0.493	47.0	LOS D	9.0	65.8	0.90	0.81	33.4
Approach		667	5.0	0.903	57.3	LOS E	21.7	158.5	0.94	0.90	30.5
All Vehicles		2443	5.0	0.903	31.9	LOS C	21.7	158.5	0.68	0.63	39.1

Level of Service (LOS) Method: Delay (HCM 2000).
Vehicle movement LOS values are based on average delay per movement
Intersection and Approach LOS values are based on average delay for all vehicle movements.
SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Movement Performance - Pedestrians								
Mov ID	Description	Demand Flow	Average Delay	Level of Service	Average Back of Queue Pedestrian	Distance	Prop. Queued	Effective Stop Rate
		ped/h	sec		ped	m		per ped
P2	NorthWest Full Crossing	53	54.3	LOS E	0.2	0.2	0.95	0.95

P1	SouthWest Full Crossing	53	54.3	LOS E	0.2	0.2	0.95	0.95
All Pedestrians		105	54.3	LOS E			0.95	0.95

Level of Service (LOS) Method: SIDRA Pedestrian LOS Method (Based on Average Delay)
Pedestrian movement LOS values are based on average delay per pedestrian movement.
Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.

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Appendix F.6 Results of Option 3A: shared through and left turn lanes with pedestrian protection for the whole walk time

INTERSECTION SUMMARY

Site: Site 1644: Albany Expressway - Massey University - Option 3A_SCL

Option 3A: shared through and left turn lane with protection for walk time
Signals - Fixed Time Cycle Time = 120 seconds (User-Given Cycle Time)

Intersection Performance - Hourly Values

Performance Measure	Vehicles		Pedestrians		Persons	
Travel Speed (Average)	37.1	km/h	1.6	km/h	36.0	km/h
Travel Distance (Total)	2482.6	veh-km/h	4.1	ped-km/h	2983.2	pers-km/h
Travel Time (Total)	66.8	veh-h/h	2.5	ped-h/h	82.8	pers-h/h
Demand Flows (Total)	2443	veh/h	105	ped/h	2932	pers/h
Percent Heavy Vehicles (Demand)	5.0	%				
Degree of Saturation	0.907		0.088			
Practical Spare Capacity	-0.8	%				
Effective Intersection Capacity	2693	veh/h				
Control Delay (Total)	24.96	veh-h/h	1.59	ped-h/h	31.54	pers-h/h
Control Delay (Average)	36.8	sec	54.3	sec	38.7	sec
Control Delay (Worst Lane)	73.8	sec				
Control Delay (Worst Movement)	73.8	sec	54.3	sec	73.8	sec
Geometric Delay (Average)	2.2	sec				
Stop-Line Delay (Average)	34.6	sec				
Idling Time (Average)	30.8	sec				
Intersection Level of Service (LOS)	LOS D		LOS E			
95% Back of Queue - Vehicles (Worst Lane)	30.3	veh				
95% Back of Queue - Distance (Worst Lane)	221.1	m				
Queue Storage Ratio (Worst Lane)	0.27					
Total Effective Stops	1756	veh/h	100	ped/h	2207	pers/h
Effective Stop Rate	0.72	per veh	0.95	per ped	0.75	per pers
Proportion Queued	0.76		0.95		0.79	
Performance Index	202.3		3.1		205.4	
Cost (Total)	1898.45	$/h	57.76	$/h	1956.21	$/h
Fuel Consumption (Total)	265.7	L/h				
Carbon Dioxide (Total)	631.9	kg/h				
Hydrocarbons (Total)	0.220	kg/h				
Carbon Monoxide (Total)	2.669	kg/h				

NOx (Total)	1.447	kg/h				

Level of Service (LOS) Method: Delay (HCM 2000).
Intersection LOS value for Vehicles is based on average delay for all vehicle movements.
Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.
SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.

Intersection Performance - Annual Values

Performance Measure	Vehicles		Pedestrians		Persons	
Demand Flows (Total)	1,172,716	veh/y	50,526	ped/y	1,407,259	pers/y
Delay	11,980	veh-h/y	762	ped-h/y	15,138	pers-h/y
Effective Stops	842,671	veh/y	48,105	ped/y	1,059,311	pers/y
Travel Distance	1,191,663	veh-km/y	1,963	ped-km/y	1,431,959	pers-km/y
Travel Time	32,088	veh-h/y	1,216	ped-h/y	39,721	pers-h/y
Cost	911,256	$/y	27,726	$/y	938,982	$/y
Fuel Consumption	127,527	L/y				
Carbon Dioxide	303,322	kg/y				
Hydrocarbons	106	kg/y				
Carbon Monoxide	1,281	kg/y				
NOx	695	kg/y				

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8001117, 6017504, NZ TRANSPORT AGENCY, NETWORK / Enterprise

MOVEMENT SUMMARY

Site: Site 1644: Albany Expressway - Massey University - Option 3A_SCL

Option 3A: shared through and left turn lane with protection for the whole walk time

Signals - Fixed Time Cycle Time = 120 seconds (User-Given Cycle Time)

Movement Performance - Vehicles

Mov ID	ODMov	Demand Flows Total	Demand Flows HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
SouthEast: Albany Expressway S											
4	L2	151	5.0	0.672	36.1	LOS D	18.5	135.2	0.82	0.77	38.3
5	T1	837	5.0	0.840	32.9	LOS C	30.3	221.1	0.88	0.83	38.7
Approach		987	5.0	0.840	33.4	LOS C	30.3	221.1	0.87	0.82	38.6
NorthWest: Albany Expressway N											
11	T1	645	5.0	0.285	7.6	LOS A	5.1	37.3	0.29	0.25	53.4
12	R2	143	5.0	0.866	73.8	LOS E	9.5	69.4	1.00	0.96	26.7
Approach		788	5.0	0.866	19.6	LOS B	9.5	69.4	0.42	0.38	45.2
SouthWest: Massey University											
1	L2	317	5.0	0.907	71.8	LOS E	22.4	163.6	1.00	1.01	27.2
3	R2	351	5.0	0.813	53.2	LOS D	20.9	152.5	0.99	0.92	31.5
Approach		667	5.0	0.907	62.0	LOS E	22.4	163.6	0.99	0.96	29.3
All Vehicles		2443	5.0	0.907	36.8	LOS D	30.3	221.1	0.76	0.72	37.1

Level of Service (LOS) Method: Delay (HCM 2000).
Vehicle movement LOS values are based on average delay per movement
Intersection and Approach LOS values are based on average delay for all vehicle movements.
SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).

HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Movement Performance - Pedestrians

Mov ID	Description	Demand Flow	Average Delay	Level of Service	Average Back of Queue		Prop. Queued	Effective Stop Rate
					Pedestrian	Distance		
		ped/h	sec		ped	m		per ped
P2	NorthWest Full Crossing	53	54.3	LOS E	0.2	0.2	0.95	0.95
P1	SouthWest Full Crossing	53	54.3	LOS E	0.2	0.2	0.95	0.95
All Pedestrians		105	54.3	LOS E			0.95	0.95

Level of Service (LOS) Method: SIDRA Pedestrian LOS Method (Based on Average Delay)
Pedestrian movement LOS values are based on average delay per pedestrian movement.
Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.

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Appendix F.7 Results of Option 3B: shared through and left turn lanes with pedestrian protection for the whole walk time and the half clearance time

INTERSECTION SUMMARY

Site: Site 1644: Albany Expressway - Massey University - Option 3B_SCL

Option 3B: shared through and left turn lane with protection for walk and half clearance intervals
Signals - Fixed Time Cycle Time = 120 seconds (User-Given Cycle Time)

Intersection Performance - Hourly Values

Performance Measure	Vehicles		Pedestrians		Persons	
Travel Speed (Average)	30.5	km/h	1.6	km/h	29.7	km/h
Travel Distance (Total)	2482.6	veh-km/h	4.1	ped-km/h	2983.2	pers-km/h
Travel Time (Total)	81.5	veh-h/h	2.5	ped-h/h	100.3	pers-h/h
Demand Flows (Total)	2443	veh/h	105	ped/h	2932	pers/h
Percent Heavy Vehicles (Demand)	5.0	%				
Degree of Saturation	1.125		0.088			
Practical Spare Capacity	-20.0	%				
Effective Intersection Capacity	2171	veh/h				
Control Delay (Total)	39.57	veh-h/h	1.59	ped-h/h	49.07	pers-h/h
Control Delay (Average)	58.3	sec	54.3	sec	60.3	sec
Control Delay (Worst Lane)	199.3	sec				
Control Delay (Worst Movement)	199.3	sec	54.3	sec	199.3	sec
Geometric Delay (Average)	2.2	sec				
Stop-Line Delay (Average)	56.1	sec				
Idling Time (Average)	51.3	sec				
Intersection Level of Service (LOS)	LOS E		LOS E			
95% Back of Queue - Vehicles (Worst Lane)	36.5	veh				
95% Back of Queue - Distance (Worst Lane)	266.3	m				
Queue Storage Ratio (Worst Lane)	0.33					
Total Effective Stops	2000	veh/h	100	ped/h	2500	pers/h
Effective Stop Rate	0.82	per veh	0.95	per ped	0.85	per pers
Proportion Queued	0.79		0.95		0.82	
Performance Index	246.9		3.1		250.0	

Cost (Total)	2367.98	$/h	57.76	$/h	2425.74	$/h
Fuel Consumption (Total)	288.8	L/h				
Carbon Dioxide (Total)	686.6	kg/h				
Hydrocarbons (Total)	0.260	kg/h				
Carbon Monoxide (Total)	2.853	kg/h				
NOx (Total)	1.530	kg/h				

Level of Service (LOS) Method: Delay (HCM 2000).
Intersection LOS value for Vehicles is based on average delay for all vehicle movements.
Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.
SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.

Intersection Performance - Annual Values

Performance Measure	Vehicles		Pedestrians		Persons	
Demand Flows (Total)	1,172,716	veh/y	50,526	ped/y	1,407,259	pers/y
Delay	18,993	veh-h/y	762	ped-h/y	23,553	pers-h/y
Effective Stops	959,787	veh/y	48,105	ped/y	1,199,850	pers/y
Travel Distance	1,191,663	veh-km/y	1,963	ped-km/y	1,431,959	pers-km/y
Travel Time	39,101	veh-h/y	1,216	ped-h/y	48,137	pers-h/y
Cost	1,136,631	$/y	27,726	$/y	1,164,357	$/y
Fuel Consumption	138,620	L/y				
Carbon Dioxide	329,546	kg/y				
Hydrocarbons	125	kg/y				
Carbon Monoxide	1,369	kg/y				
NOx	734	kg/y				

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MOVEMENT SUMMARY

Site: Site 1644: Albany Expressway - Massey University - Option 3B_SCL

Option 3B: shared through and left turn lane with protection for walk and half clearance intervals

Signals - Fixed Time Cycle Time = 120 seconds (User-Given Cycle Time)

Movement Performance - Vehicles

Mov ID	ODMov	Demand Flows Total	Demand Flows HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
SouthEast: Albany Expressway S											
4	L2	151	5.0	0.714	41.9	LOS D	18.4	134.6	0.89	0.81	36.1
5	T1	837	5.0	0.893	39.7	LOS D	36.5	266.3	0.94	0.93	36.1
Approach		987	5.0	0.893	40.0	LOS D	36.5	266.3	0.93	0.91	36.1
NorthWest: Albany Expressway N											
11	T1	645	5.0	0.298	9.3	LOS A	5.9	43.3	0.34	0.29	52.1
12	R2	143	5.0	1.125	199.3	LOS F	16.7	121.9	1.00	1.36	13.9
Approach		788	5.0	1.125	43.8	LOS D	16.7	121.9	0.46	0.49	34.8
SouthWest: Massey University											
1	L2	317	5.0	1.082	164.6	LOS F	34.6	252.6	1.00	1.30	16.1
3	R2	351	5.0	0.752	46.3	LOS D	19.1	139.4	0.95	0.88	33.5
Approach		667	5.0	1.082	102.5	LOS F	34.6	252.6	0.97	1.08	22.1
All Vehicles		2443	5.0	1.125	58.3	LOS E	36.5	266.3	0.79	0.82	30.5

Level of Service (LOS) Method: Delay (HCM 2000).
Vehicle movement LOS values are based on average delay per movement
Intersection and Approach LOS values are based on average delay for all vehicle movements.
SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.

Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Movement Performance - Pedestrians								
Mov ID	Description	Demand Flow	Average Delay	Level of Service	Average Back of Queue		Prop. Queued	Effective Stop Rate
					Pedestrian	Distance		
		ped/h	sec		ped	m		per ped
P2	NorthWest Full Crossing	53	54.3	LOS E	0.2	0.2	0.95	0.95
P1	SouthWest Full Crossing	53	54.3	LOS E	0.2	0.2	0.95	0.95
All Pedestrians		105	54.3	LOS E			0.95	0.95

Level of Service (LOS) Method: SIDRA Pedestrian LOS Method (Based on Average Delay)
Pedestrian movement LOS values are based on average delay per pedestrian movement.
Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.

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Appendix F.8 Results of Option 3C: shared through and left turn lanes pedestrian protection for the whole walk time and the full clearance time

INTERSECTION SUMMARY

Site: Site 1644: Albany Expressway - Massey University - Option 3C_SCL

Option 3C: shared through and left turn lane protection for walk and full clearance intervals
Signals - Fixed Time   Cycle Time = 120 seconds (User-Given Cycle Time)

Intersection Performance - Hourly Values						
Performance Measure	Vehicles		Pedestrians		Persons	
Travel Speed (Average)	13.7	km/h	1.6	km/h	13.6	km/h
Travel Distance (Total)	2482.6	veh-km/h	4.1	ped-km/h	2983.2	pers-km/h
Travel Time (Total)	180.6	veh-h/h	2.5	ped-h/h	219.2	pers-h/h
Demand Flows (Total)	2443	veh/h	105	ped/h	2932	pers/h
Percent Heavy Vehicles (Demand)	5.0	%				
Degree of Saturation	2.009		0.088			
Practical Spare Capacity	-55.2	%				
Effective Intersection Capacity	1216	veh/h				
Control Delay (Total)	138.70	veh-h/h	1.59	ped-h/h	168.03	pers-h/h
Control Delay (Average)	204.4	sec	54.3	sec	206.3	sec
Control Delay (Worst Lane)	988.6	sec				
Control Delay (Worst Movement)	988.6	sec	54.3	sec	988.6	sec
Geometric Delay (Average)	2.2	sec				
Stop-Line Delay (Average)	202.2	sec				
Idling Time (Average)	194.2	sec				
Intersection Level of Service (LOS)	LOS F		LOS E			
95% Back of Queue - Vehicles (Worst Lane)	85.9	veh				
95% Back of Queue - Distance (Worst Lane)	627.1	m				
Queue Storage Ratio (Worst Lane)	0.77					
Total Effective Stops	2471	veh/h	100	ped/h	3065	pers/h
Effective Stop Rate	1.01	per veh	0.95	per ped	1.05	per pers
Proportion Queued	0.79		0.95		0.82	

Performance Index	438.0		3.1		441.1	
Cost (Total)	5362.71	$/h	57.76	$/h	5420.47	$/h
Fuel Consumption (Total)	416.0	L/h				
Carbon Dioxide (Total)	987.3	kg/h				
Hydrocarbons (Total)	0.503	kg/h				
Carbon Monoxide (Total)	3.930	kg/h				
NOx (Total)	1.817	kg/h				

Level of Service (LOS) Method: Delay (HCM 2000).
Intersection LOS value for Vehicles is based on average delay for all vehicle movements.
Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.
SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.

Intersection Performance - Annual Values

Performance Measure	Vehicles		Pedestrians		Persons	
Demand Flows (Total)	1,172,716	veh/y	50,526	ped/y	1,407,259	pers/y
Delay	66,577	veh-h/y	762	ped-h/y	80,654	pers-h/y
Effective Stops	1,185,937	veh/y	48,105	ped/y	1,471,229	pers/y
Travel Distance	1,191,663	veh-km/y	1,963	ped-km/y	1,431,959	pers-km/y
Travel Time	86,685	veh-h/y	1,216	ped-h/y	105,238	pers-h/y
Cost	2,574,101	$/y	27,726	$/y	2,601,827	$/y
Fuel Consumption	199,671	L/y				
Carbon Dioxide	473,881	kg/y				
Hydrocarbons	241	kg/y				
Carbon Monoxide	1,887	kg/y				
NOx	872	kg/y				

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INTERSECTION 6

MOVEMENT SUMMARY

Site: Site 1644: Albany Expressway - Massey University -Option 3C_SCL

Option 3C: shared through and left turn lane protection for walk and full clearance intervals

Signals - Fixed Time Cycle Time = 120 seconds (User-Given Cycle Time)

Movement Performance - Vehicles

Mov ID	ODMov	Demand Flows Total veh/h	Demand Flows HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	95% Back of Queue Distance m	Prop. Queued	Effective Stop Rate per veh	Average Speed km/h
SouthEast: Albany Expressway S											
4	L2	151	5.0	0.717	48.4	LOS D	16.4	119.5	0.93	0.84	33.7
5	T1	837	5.0	0.896	38.1	LOS D	39.4	287.7	0.93	0.93	36.8
Approach		987	5.0	0.896	39.6	LOS D	39.4	287.7	0.93	0.91	36.3
NorthWest: Albany Expressway N											
11	T1	645	5.0	0.294	8.7	LOS A	5.7	41.3	0.32	0.28	52.5
12	R2	143	5.0	1.876	868.9	LOS F	36.8	268.7	1.00	2.08	3.9
Approach		788	5.0	1.876	164.9	LOS F	36.8	268.7	0.45	0.61	16.2
SouthWest: Massey University											
1	L2	317	5.0	2.009	988.6	LOS F	85.9	627.1	1.00	2.45	3.5
3	R2	351	5.0	0.772	48.3	LOS D	19.6	143.4	0.96	0.89	32.9
Approach		667	5.0	2.009	494.8	LOS F	85.9	627.1	0.98	1.63	6.6
All Vehicles		2443	5.0	2.009	204.4	LOS F	85.9	627.1	0.79	1.01	13.7

Level of Service (LOS) Method: Delay (HCM 2000).
Vehicle movement LOS values are based on average delay per movement

Intersection and Approach LOS values are based on average delay for all vehicle movements.
SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Movement Performance - Pedestrians

Mov ID	Description	Demand Flow	Average Delay	Level of Service	Average Back of Queue		Prop. Queued	Effective Stop Rate
					Pedestrian	Distance		
		ped/h	sec		ped	m		per ped
P2	NorthWest Full Crossing	53	54.3	LOS E	0.2	0.2	0.95	0.95
P1	SouthWest Full Crossing	53	54.3	LOS E	0.2	0.2	0.95	0.95
All Pedestrians		105	54.3	LOS E			0.95	0.95

Level of Service (LOS) Method: SIDRA Pedestrian LOS Method (Based on Average Delay)
Pedestrian movement LOS values are based on average delay per pedestrian movement.
Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.

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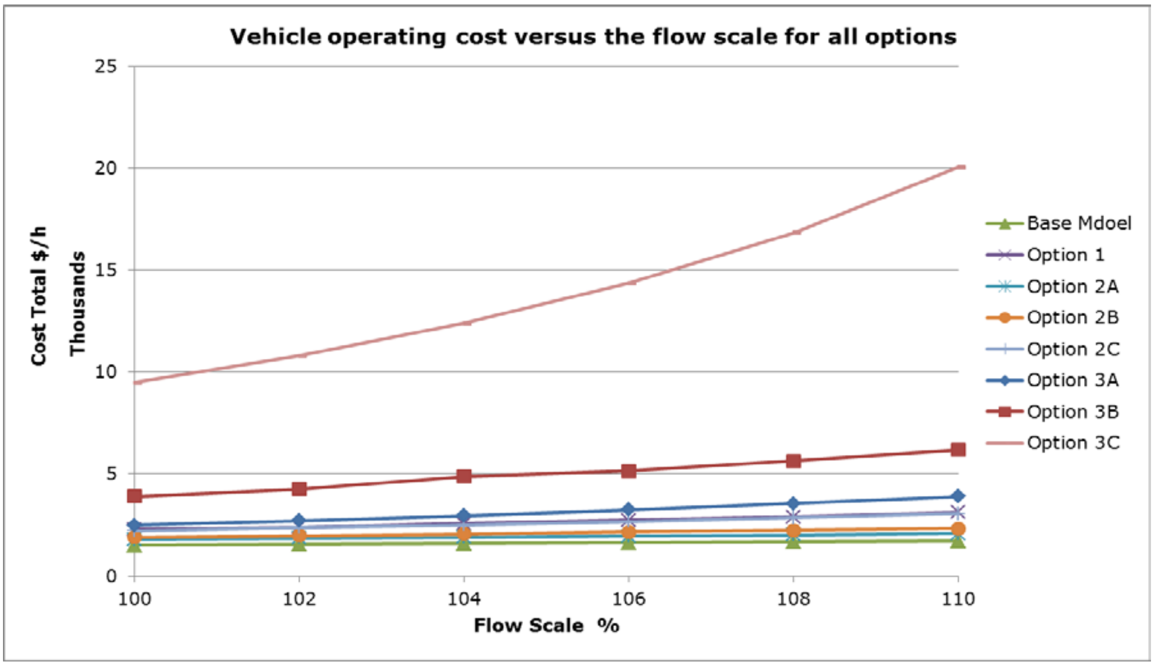
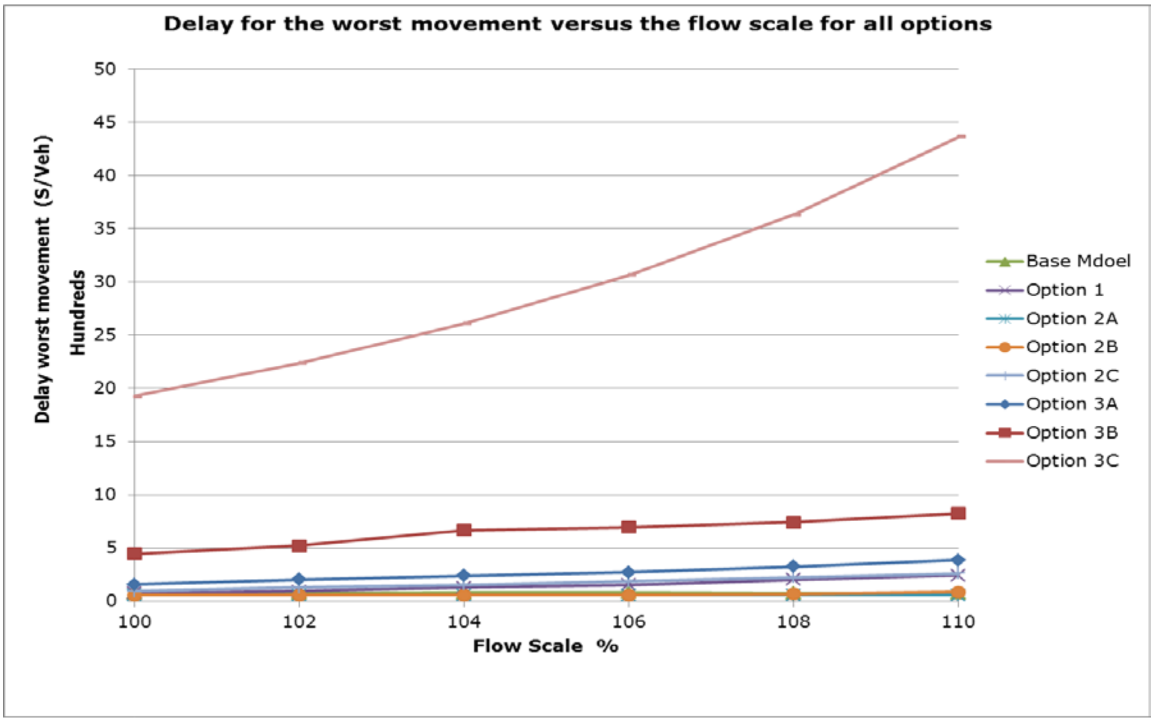
Appendix G Results of Method B1 (MB1): increasing the left turn traffic volumes

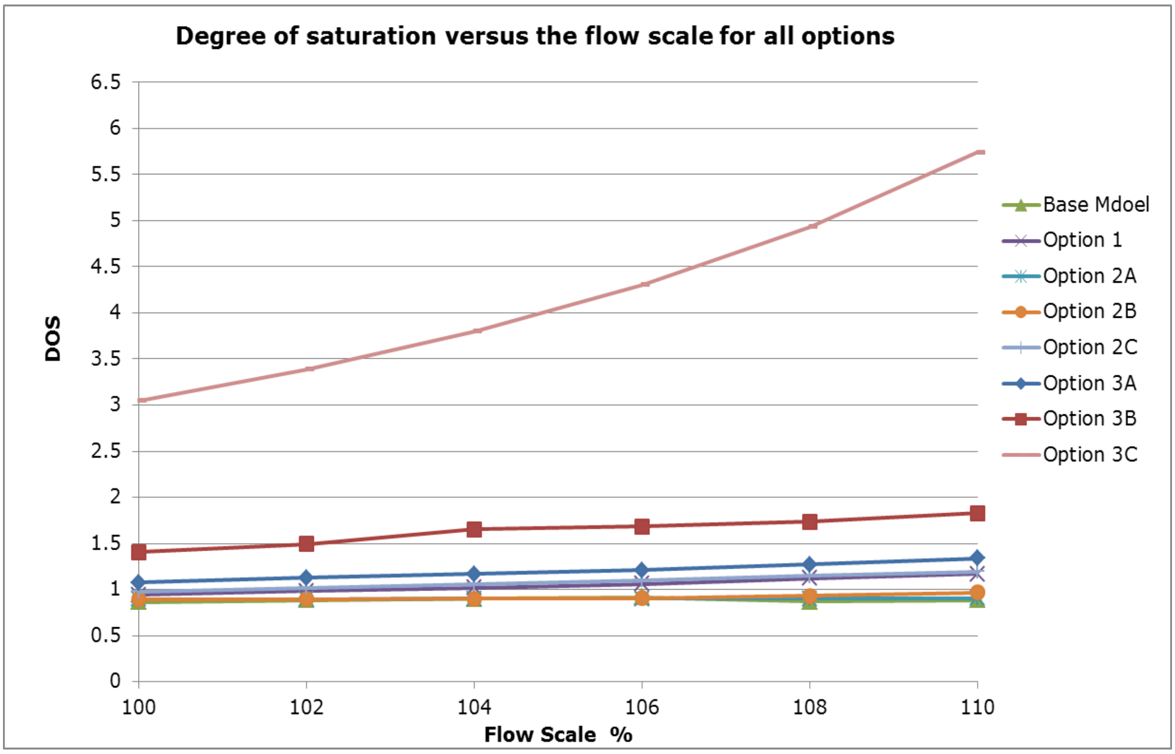
Appendix G.1 Results of MB1 - vehicle operating cost



Appendix G.2 Results of MB1 - delay for the worst movement

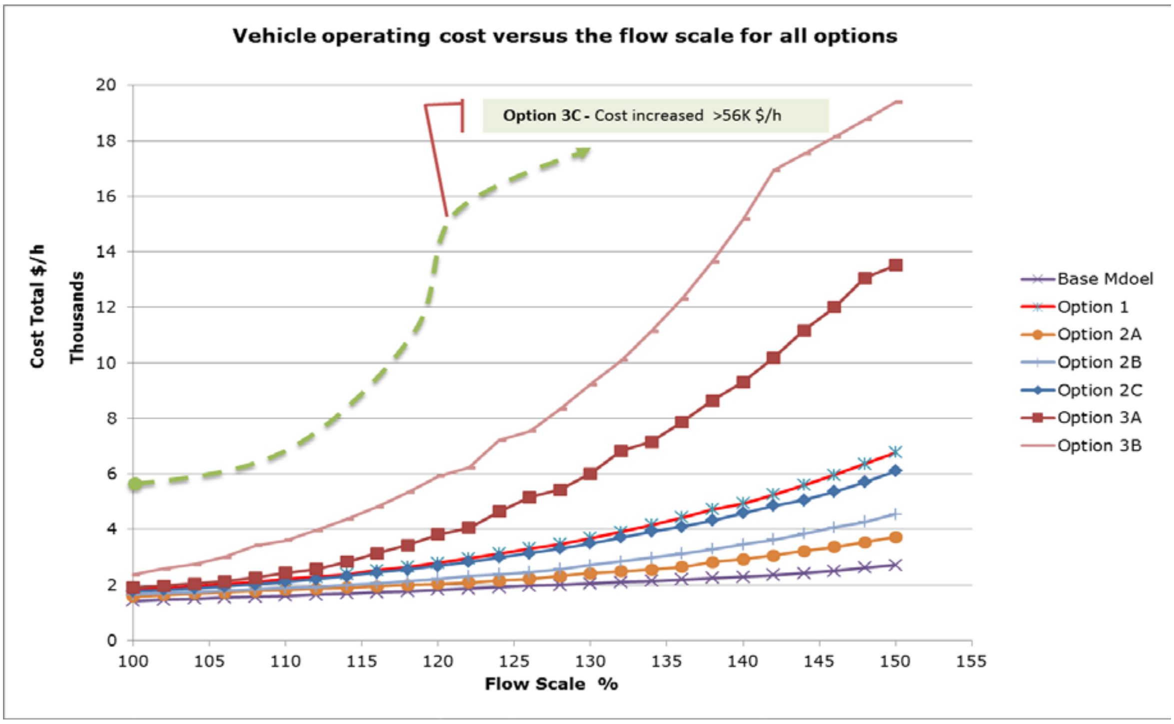


Appendix G.3 Results of MB1 - degree of saturation

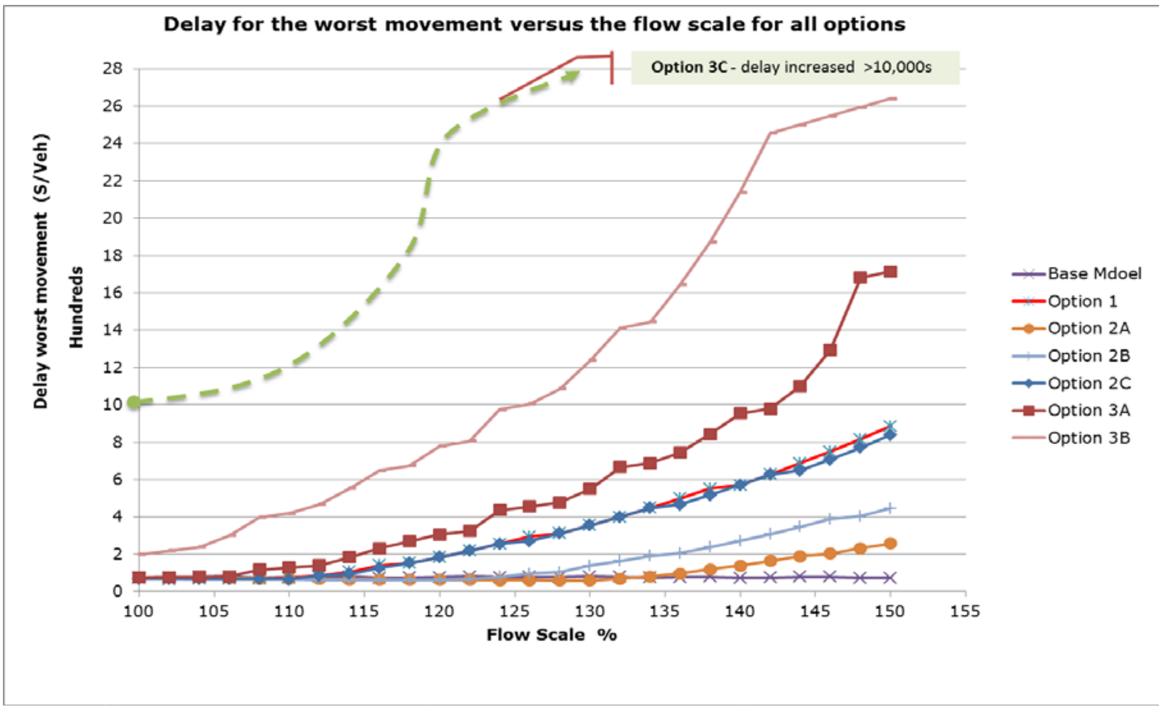


Appendix H Results of Method B2 (MB2): increasing the intersection traffic volumes

Appendix H.1 Results of MB2 - vehicle operating cost



Appendix H.2 Results of MB2-delay for the worst movement



Appendix H.3 Results of MB2 - degree of saturation